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FEBRUARY 1961 VOL. LVI. NO. 2

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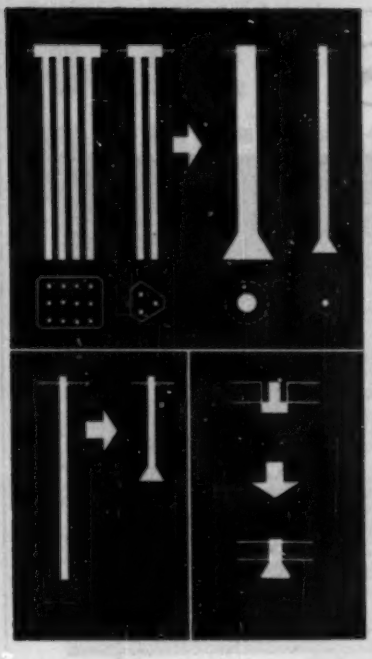

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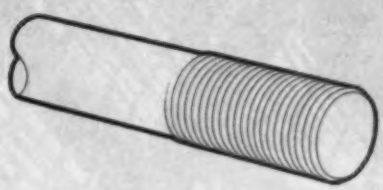

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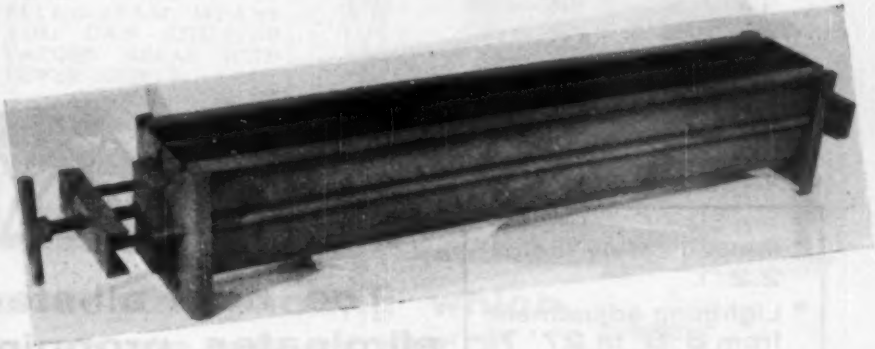

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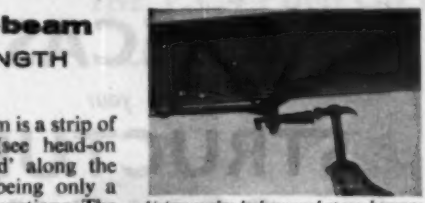

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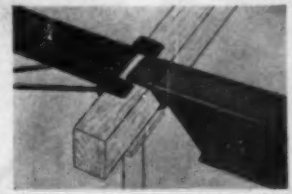

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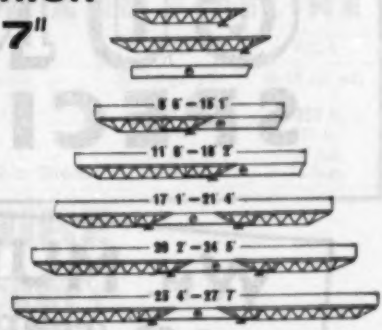

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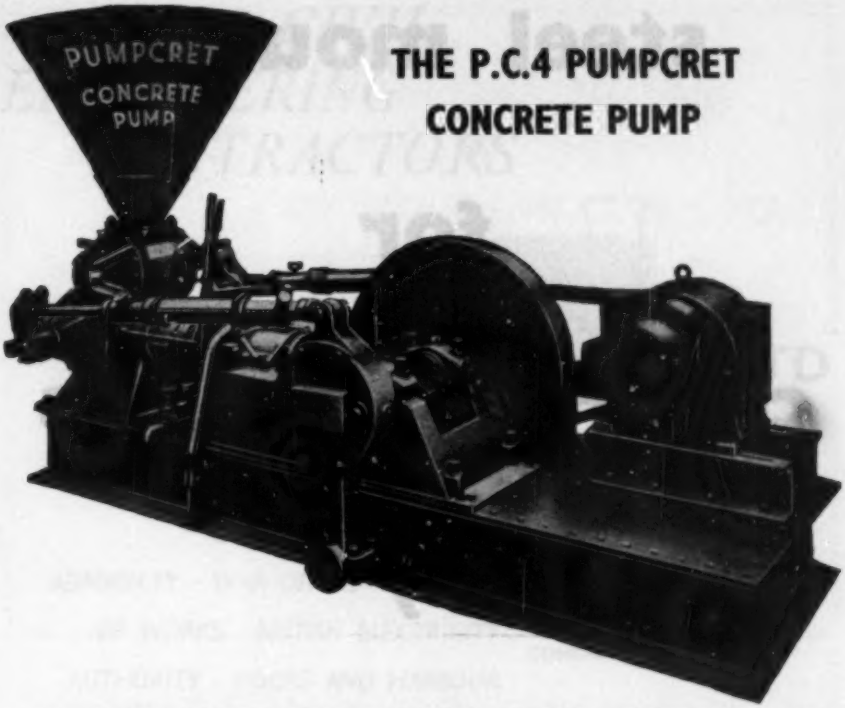

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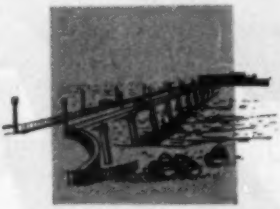

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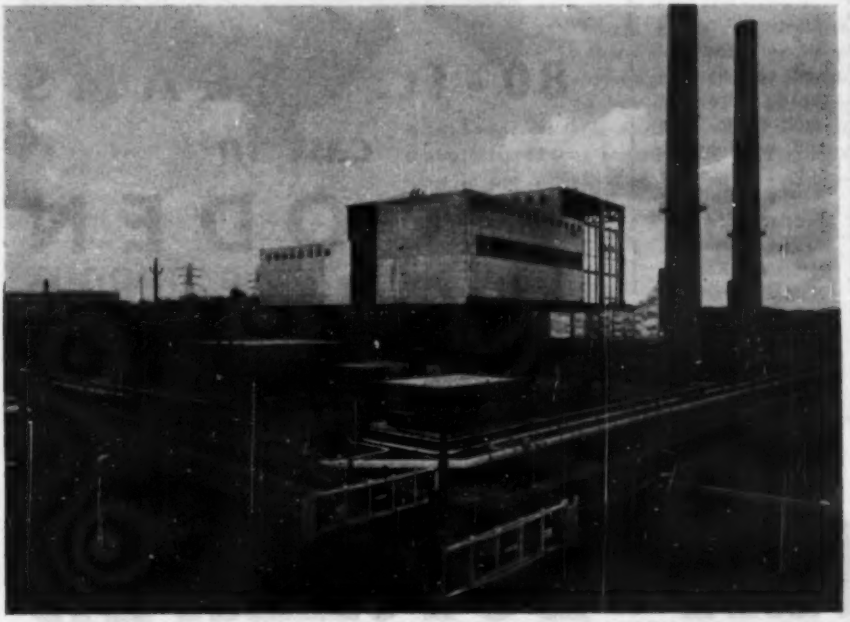

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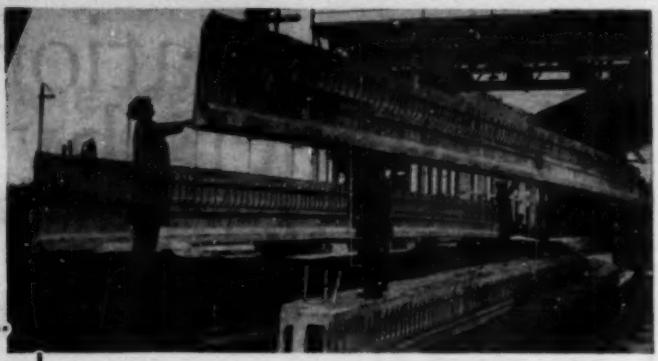

Illustration above shows some of the post-stressed bridge beams, each 80 ft. long and weighing 35 tons, produced by Anglian Building Products Ltd., for British Railways, Midland Region. Main Contractors: Messrs. Leonard Fairclough Ltd.

80-ft. BEAMS
cast in
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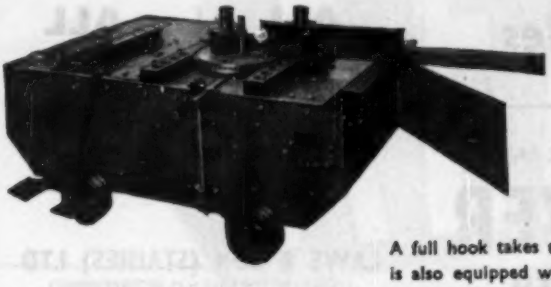

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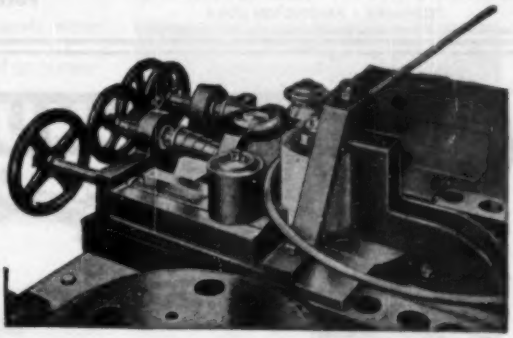
We supply, at extra cost, an appliance (illustrated here) for use with RAS.40 and ARD.50 models for the bending of hoops and spirals. Bends radii of 11 in. and upwards and to any pitch of spiral.



Our Bending Equipment also includes the ARD.50 model—a double-disc machine for bending bars up to 2 in. diameter. We also supply, at extra cost, formers and backrests for special steel such as "Square Grip," "Twisteel," etc., which can be fitted to the ARD.50 and RAS.40 models. Full details are available on request. All machines are available for Sale or Hire.

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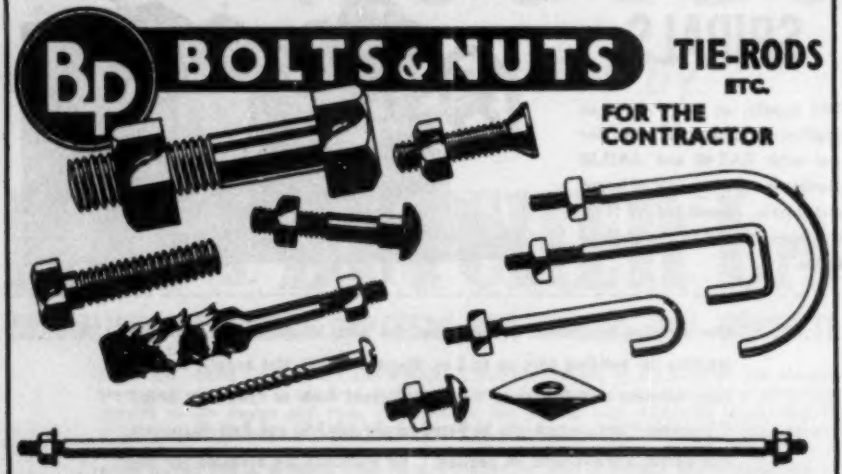

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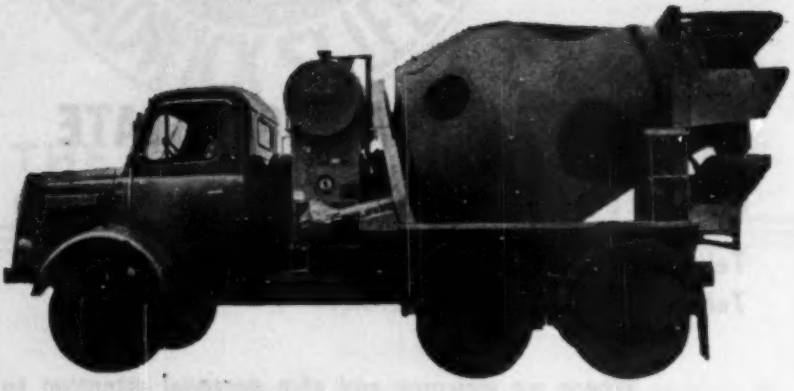

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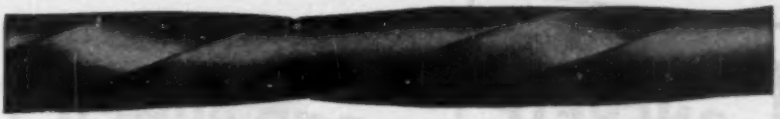

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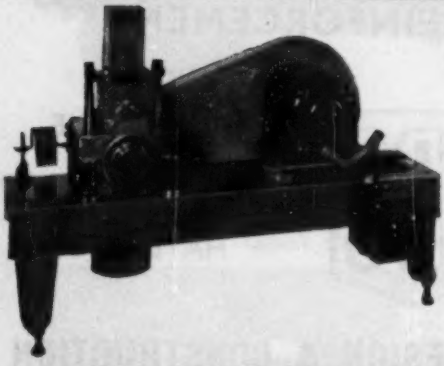

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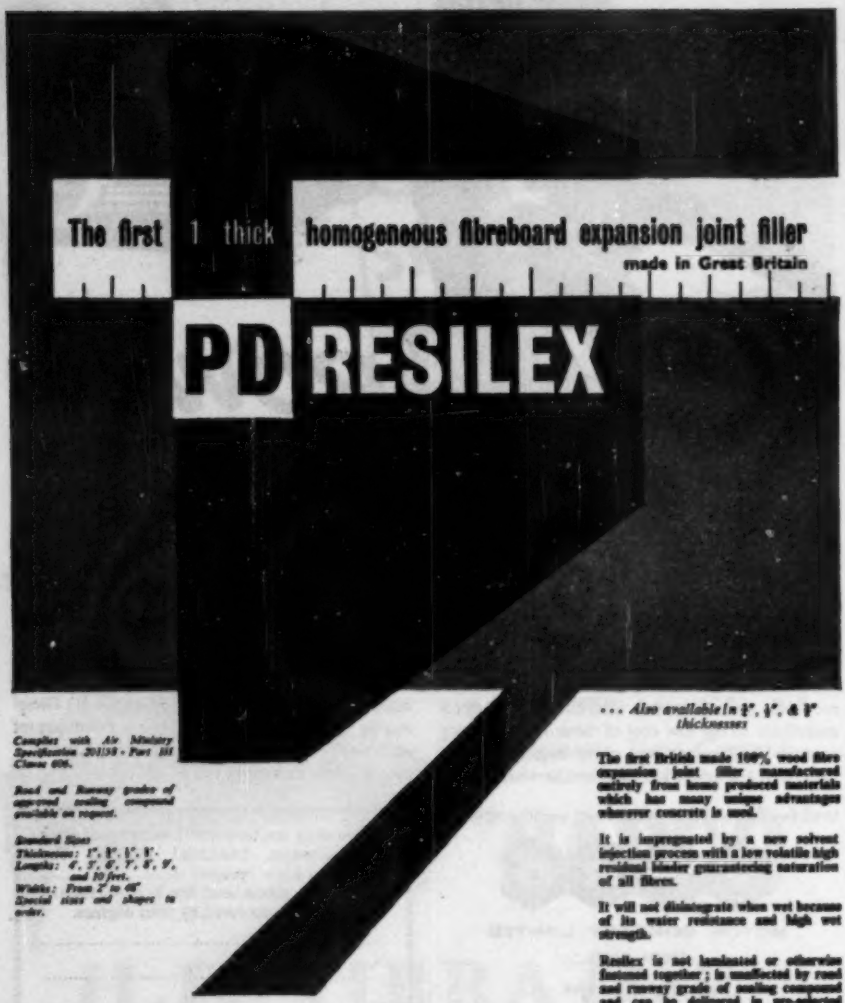

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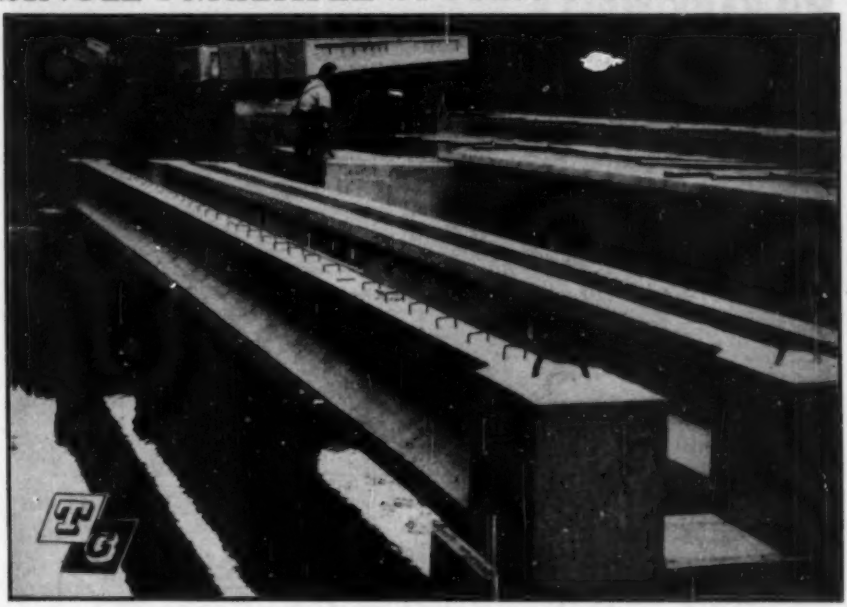

New road bridge during construction on A.50 at Foston, Derbyshire, showing 40-ft. prestressed beams (pretensioned steel) designed to M.O.T. requirements. Main Contractors: D. T. Bullock & Co. Ltd., Aldridge, Staffs. By courtesy of the County Surveyor of Derbyshire—S. Mehew, Esq., O.B.E., B.Sc., M.I.C.E., A.M.T.P.I.

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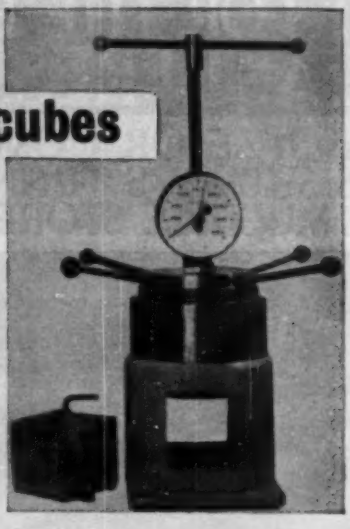

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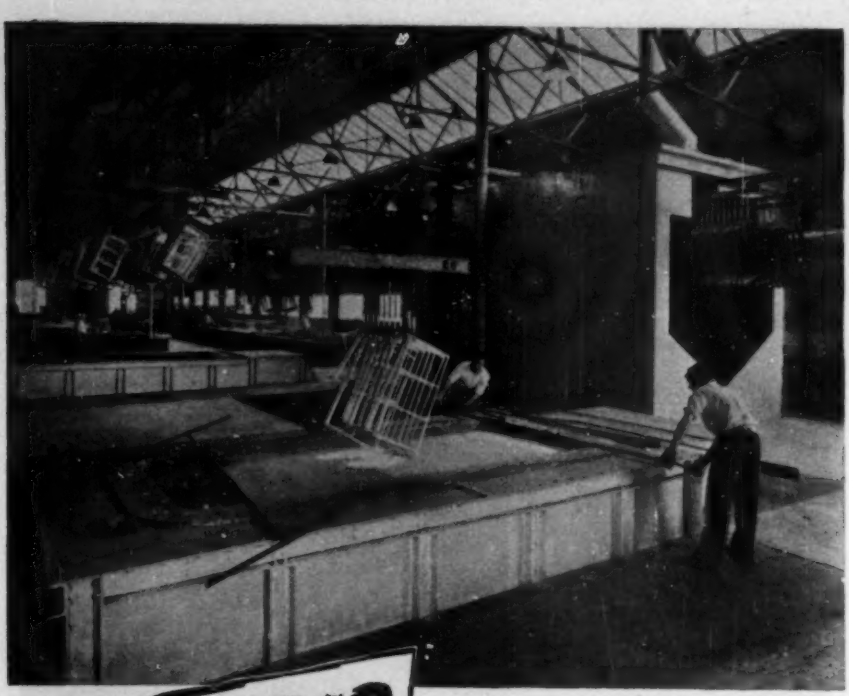

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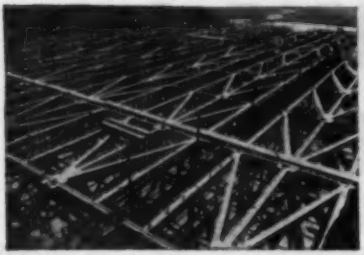

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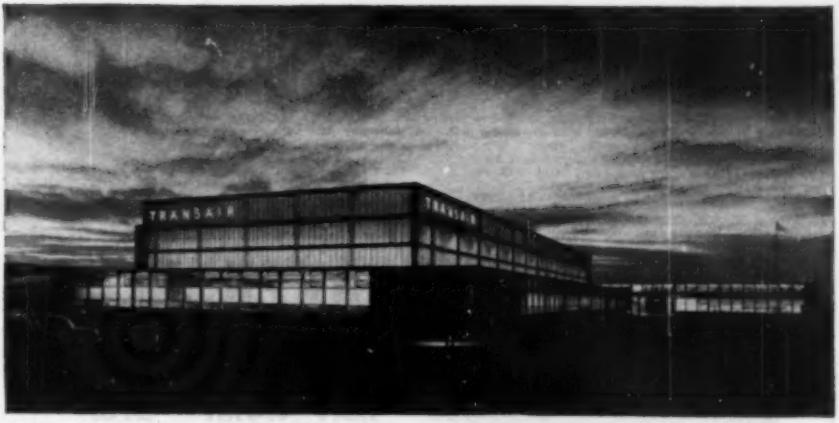

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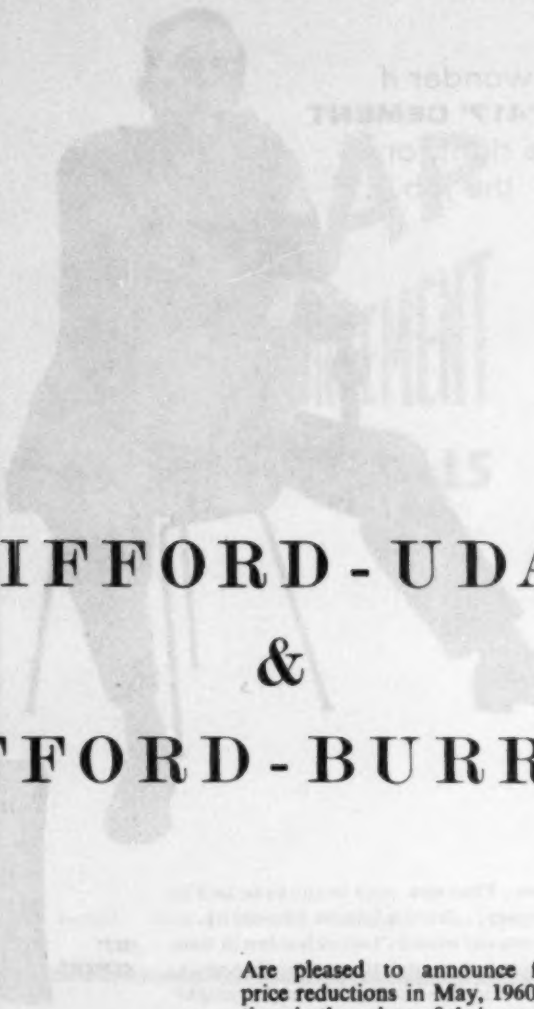

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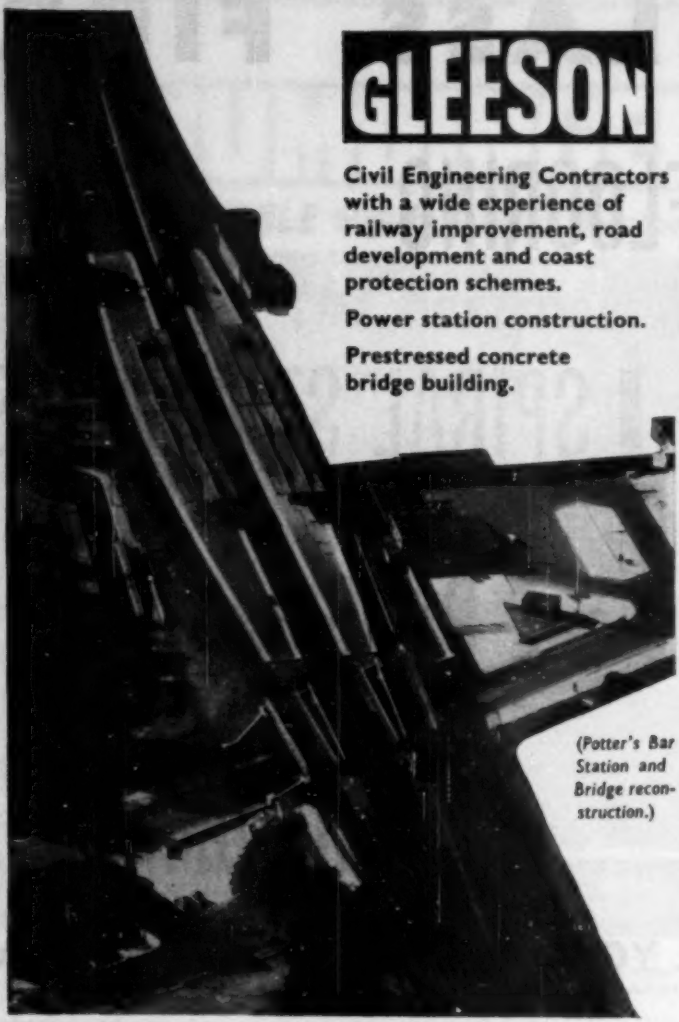

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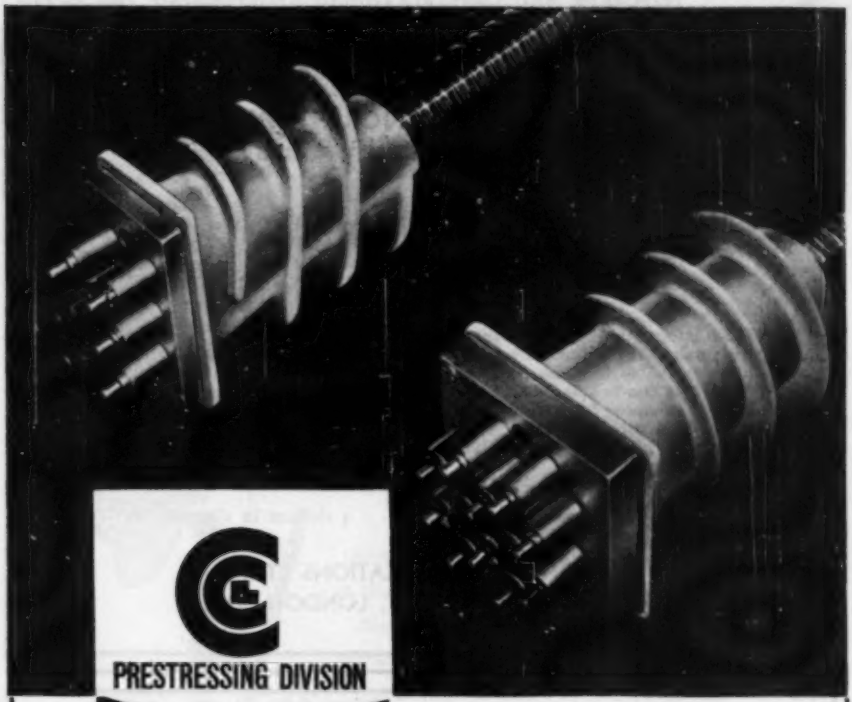

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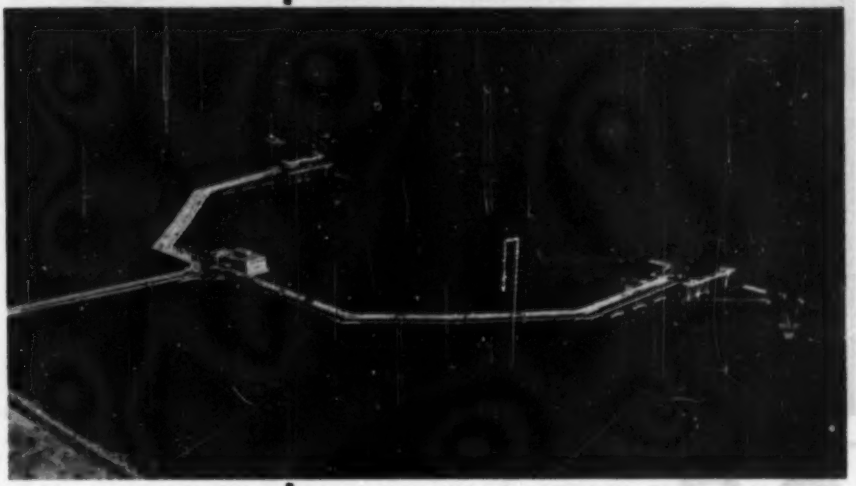

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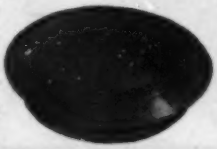

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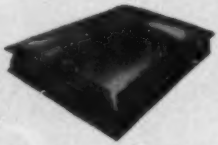

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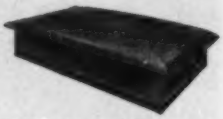

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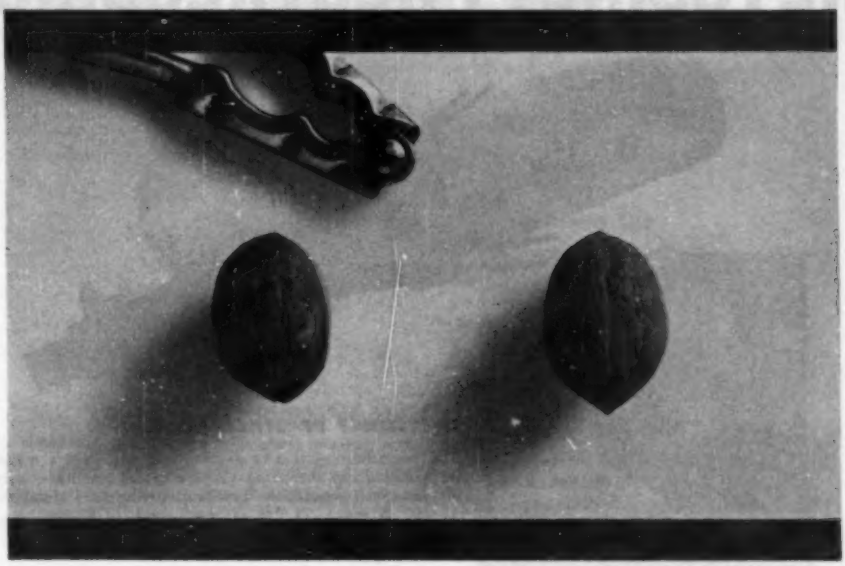

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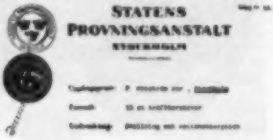

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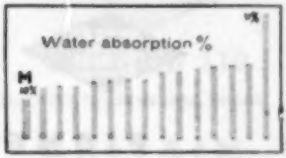

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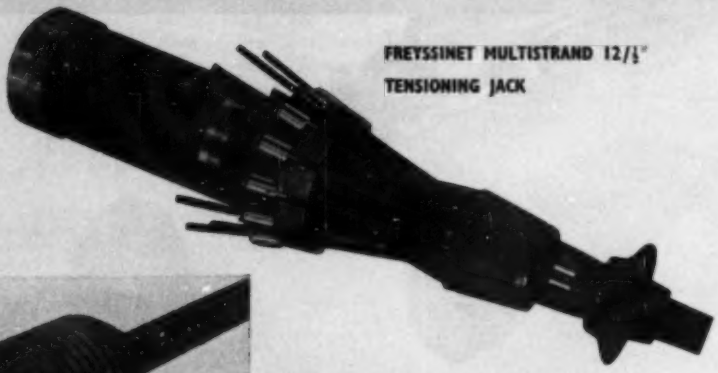

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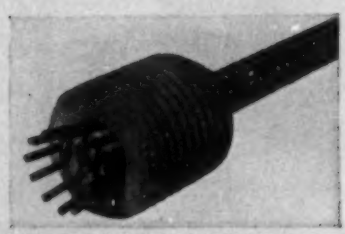

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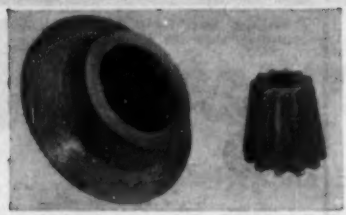

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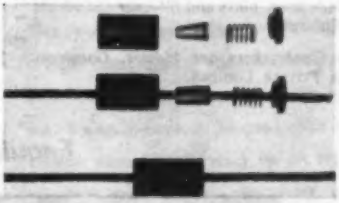

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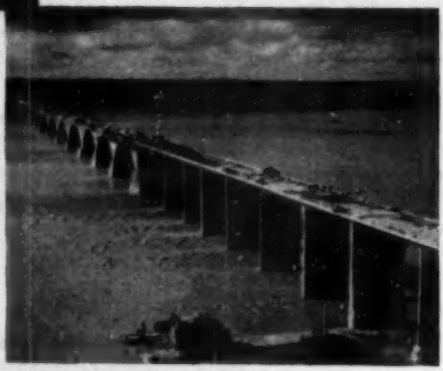

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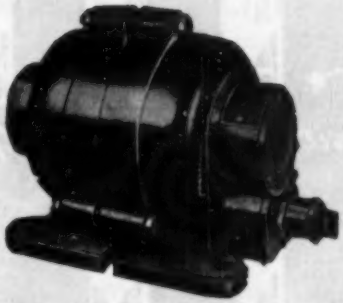

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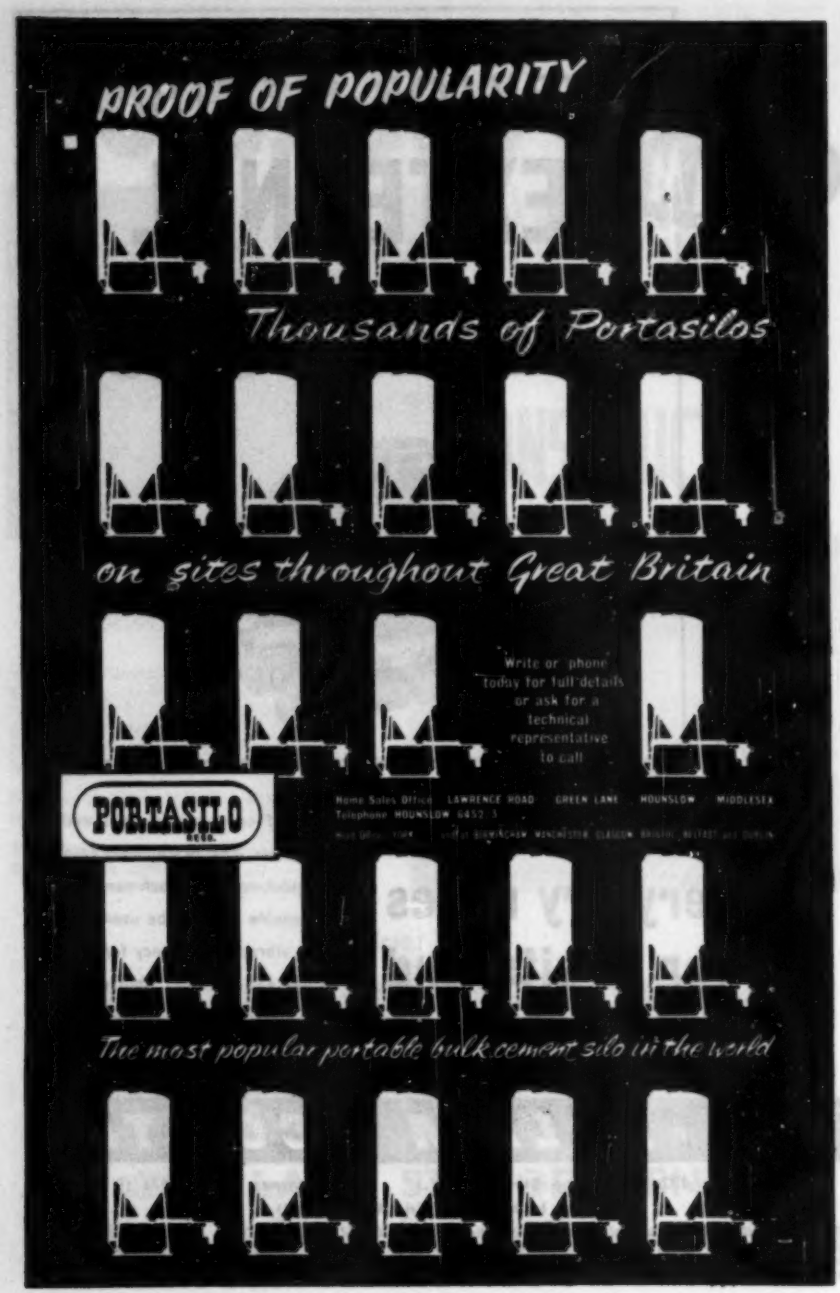
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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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Volume LVI, No. 2. LONDON, FEBRUARY, 1961.

EDITORIAL NOTES

Structural Concrete with Lightweight Aggregate.

MUCH interest is being taken currently in the use of lightweight concrete as a structural material. This interest has been aroused not only by the efforts of firms interested commercially in the material but by the publicity which is being given to research work carried out by the Building Research Station* on cellular concrete and concrete made with lightweight aggregate. Some correlation of this research work with practice was attained at a meeting arranged by the Building Research Station and held in December last at the Building Centre in London.

The advantages of lightweight concrete for structural members are obvious in so far that the dead weight of the structure is likely to be less than that of ordinary reinforced concrete and that higher degrees of thermal and acoustic insulation and resistance to fire are also obtained. Another aspect is the concern which has been expressed in some quarters regarding the probable shortage of natural concrete aggregates in this country in the not too distant future. Indications of the approach of this state include the rapidity with which available beds of gravel and sand are being excavated and the fact that many of the sites of such beds are generally either good agricultural land or " desirable " building sites. Some seventy million tons of gravel and sand were used in Britain in 1959 and the statistics available for 1960 seem to show that for last year this amount is likely to be exceeded. Sources of crushed stone suitable for concrete aggregates are not always near the place where they are required for use and the costs of quarrying, preparing and transporting are continually increasing.

For these and similar reasons more intensive attention is being directed to the technical problems and economic factors appertaining to structural concrete using other forms of aggregates, such as treated natural and artificial materials,

* " The Use of Lightweight Concrete for Reinforced Concrete Construction." By A. Short. (*Reinforced Concrete Review*, September 1959.)
" The Structural Use of Aerated Concrete." By A. Short and W. Kinniburgh. (*The Structural Engineer*, January 1961.)
" Materials for Concrete." (Building Research Station Digest, No. 5, December 1960.)
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many of which are by-products of some industrial processes. The use of lightweight concrete for structural purposes is by no means new. Such concrete in combination with ordinary denser concrete around the reinforcement has been the basis of some proprietary methods of construction for several decades. Similarly the use of reinforced cellular concrete dates, in this country at least, from at least ten years or more ago and, based on Swedish experience, is being developed further as regards reinforced slabs if not yet in the form of main structural members.

Even concrete made with lightweight aggregate in the form of sintered clay and having sufficient strength to be suitable for prestressed concrete can no longer be considered to be a newcomer. Neither can the use of load-bearing concrete with lightweight aggregate and without reinforcement be classed as a new development in this country, although the most recent example, in which an expanded-clay aggregate from Denmark is being used in the construction of a thirteen-storey block of flats for the London County Council, is an almost startling application of the method.

From the view-point of the design engineer, one of the major drawbacks of lightweight concrete has been that lightness of weight is accompanied generally by low strength, but even now that the strength can be increased with only a modest increase of weight, the use of lightweight aggregates in structural reinforced concrete raises questions regarding the suitability of the material in other respects. Doubtful factors include the degree of creep and shrinkage, the liability to excessive deflection, and the effectiveness of bond with, and protection afforded to, the steel reinforcement. It is to establish reliable data relating to these and similar technical factors that so much research has been directed to structural reinforced lightweight concrete.

The variety of lightweight aggregates available makes most generalisations about structural lightweight concrete unreliable, and some contradictory results may be obtained. It is generally thought that lightweight concrete is subjected to greater shrinkage than ordinary concrete but experience, especially in the case of the flats mentioned in the foregoing, seems to show that shrinkage cracks can be avoided if care is taken in proportioning and compacting the material. The recommendations of the American Concrete Institute for lightweight concrete having a crushing strength of not less than 2000 lb. per square inch suggest that a slump of 4 in. may be required to obtain sufficient workability, but again, with an expanded-clay aggregate, a concrete of comparable strength having a slump of 1 in. seems to be most suitable if a plasticiser is used.

The results of the investigations by the Building Research Station show that the modulus of elasticity is about half that of ordinary concrete of the same compressive strength. This fact generally leads to members exhibiting a deflection that may be ten to fifty per cent. greater than that of similar members of ordinary reinforced concrete; for this reason it is recommended that the ratio of span to depth of beams and slabs of lightweight concrete should not exceed three-quarters of the ratios given in B.S. Code No. 114. Some of the design recommendations in this Code may apply directly to structural concrete with lightweight aggregates, but others are by no means applicable. The bond of lightweight concrete to steel bars is less than that of ordinary concrete and half only of the permissible bond stresses recommended in the Code should be allowed, and all reinforcement

subjected to tension should be provided with hooks or other forms of end-anchorage. Although the ratio of the tensile and compressive strengths of lightweight-aggregate concrete may be slightly greater than that of ordinary concrete of the same strength, it is advised that the permissible shearing stresses in slabs should be three-quarters of those recommended in the Code and, in the case of beams, the shearing forces should be resisted entirely by reinforcement.

The durability of lightweight concrete appears to depend not so much on the quality of the aggregate as on the alkali-content of the cement and the quality of the cement-sand matrix. The susceptibility of steel reinforcement to corrosive influences in the more porous material has lead to the practice, which is obligatory in some European countries, of coating the bars with cement slurry or with cement-latex or other material.

The tensile stress in mild steel reinforcement embedded in lightweight concrete may be the same as that in ordinary concrete, but it is recommended that the stress in high-tensile steel or high-yield-stress bars should be limited to 25,000 lb. per square inch. It has been reported that cracking of lightweight concrete beams may be more severe than with ordinary concrete, the cracks being fewer but wider; in major members at least, the use of deformed bars might be an advantage, but evidence in support of this does not seem to be available.

There is undoubtedly a future for lightweight structural concrete, and no doubt in the course of time, more definite rules for the design of members of such concrete will be formulated. What is apparent at present is that, apart from design, the success or otherwise of structural concrete with lightweight aggregate depends on the preparation and proportioning of the materials (especially the water) and the mixing and compaction of the concrete to the right degree, and the probable provision of a plasticiser or air-entraining agent. Although the results of laboratory tests may help, experience of actual works seems to be the best guide. The characteristics of lightweight concrete seem to be another indication that making concrete is a craft. The making of lightweight concrete should not be left to unskilled or untrained men, otherwise troubles may occur which will result in a potentially good material getting into bad repute.

Construction Costs.

A FEATURE of this journal, almost from its inception until two years ago, was the publication of a summary of current prices of materials and labour, and the combined costs of concrete work. Alterations in the cost of labour brought about by the reduction of the number of hours in a working week in the civil engineering and building industries, variations in the price of steel and cement, and such factors have made it difficult until recently to compile reliable data. Now that there is some reasonable stability in these matters, at least until the increases in October next come into operation, the publication of the page entitled " Data for Pricing Reinforced Concrete " has been resumed in this number.

The prices are not applicable throughout the country, but they are put forward as providing a reasonable basis on which engineers may estimate the probable cost of works and, in particular, the comparative costs of alternative schemes. It is not expected that contractors would use this data to prepare firm tenders for works.

Book Reviews.

"Practical Problems in Foundations." By H. R. Reynolds. (London: Crosby Lockwood & Son, Ltd. 1960. 25s.).

THE author covers a very wide field in a book of less than two-hundred pages and the many subjects are therefore treated rather superficially. Many references to well-known structures, including the Pons Sublicius and the Leaning Tower of Pisa, are included. It must be presumed that serious readers of any work on foundations have access to the codes of practice for "Site Investigations" and "Foundations" but the author does not refer to either. Many statements put forward as general truths are true only in certain circumstances, while others are in error, such as "... all piles driven in Great Britain ... in recent years have light sheet-metal shoes ..." Some opinions seem to contradict others, such as "... the pneumatic process compares unfavourably as regards cost ... it has the advantage of the rapidity and ease with which the caisson can be sunk ...."

Some of the calculations are not clear. One diagram shows a load of 550 tons on an area of 20 ft. by 22 ft., yet the text states "The upward pressure p is between 500 and 1500 lb. per square foot." Some of the calculated results are improbable; one example is a pair of 12-in. diameter bored piles every 2 ft. under the party wall of a two-storey house.

Some of the slips are probably due to failure to make sure that the reader understands what the author has in mind. Strict editing of this first edition could produce a good second edition.—G. P. M.

"Gas- und Schaumbeton. Verankerung der Bewehrung und Kriechen von bewehrten Platten." Deutscher Ausschuss für Stahlbeton. Pamphlet No. 136. By H. Schäffler. (Berlin: Wilhelm Ernst & Sohn. 1960. Price 21 D.M.)

THIS pamphlet, issued on behalf of the German Select Committee for Reinforced Concrete, describes tests on the bond of reinforcement in steam-cured ærated concrete and on the creep of slabs of this material. The tests show that the bond produced by coating the reinforcement with cement mortar is stronger than that produced by a bituminous coating and that the bond stress is to some extent dependent on the strength of the concrete although its rate of increase is not proportional to the strength. The influence of the diameter and embedded length of the reinforcement was not so marked as for ordinary concrete. The effectiveness of the bond depends to a considerable extent on the resistance of the reinforcement to slipping, in spite of the limited strength of the concrete. The deformation of ærated concrete slabs due to creep is about the same as that of slabs of ordinary concrete. The increase of deflection of ærated concrete slabs is stated to be more serious and in some cases was doubled in the course of a year and had not ceased to increase after two years.

"Can Man Plan?" By F. J. Osborn. (London: George Harrap & Co., Ltd. 1959. Price 12s. 6d.)

THIS book contains about a hundred of Sir Frederic Osborn's short poems, most in a light vein, and takes its title from one of them. A fair sample of these verses is that entitled "August Perret: A Cockney Tribute", which runs

"The neat job what Nash done in plastering London
Got bonked by the bombs and a lot of it undone.
So old Perret cashed in on the rise in the Bonk-rate:
He found us all plastered, and left us all concrate."

Books Received.

"Structurally Reinforced Concrete Pavements." By A. Losberg. (Gotenburgh: Elanders Boktryckeri Aktiebolag. In English. 1960. No price stated.)

"Steel Designers' Manual." By C. S. Gray, L. E. Kent, W. A. Mitchell and G. B. Godfrey. (London: Crosby Lockwood and Son, Ltd. Second Edition. 1960. Price 50s.)

This edition has been revised extensively to conform to the 1959 issue of British Standard No. 449.

"Le Probleme du Broyage et son Evolution: L'aptitude des Matériaux au Broyage." By R. Guillot. (Paris: Eyrolles and Gauthier-Villars. In French. 1960. Price 29.35 NF.)

Position of Planes of Contraflexure due to Wind on Multiple-storey Buildings.

By P. M. HEWLETT.

In calculating the bending moments, due to wind, on the columns of a multiple-storey building, the following simple method can be used to determine the limits between which the planes of contraflexure in the columns must lie, and thence the greatest bending moment on each column.

Most structural frames can be considered to be composed of symmetrical frames such as that shown in *Fig.* 1. If W is the horizontal force due to wind at the level of each floor, then the force at roof level is $\frac{1}{2}W$ and the shearing force on each column due to wind is rW, in which r is the number of storeys from the top to the level under consideration counting the top storey as $\frac{1}{2}$. For simplicity a frame is considered in which the storey-height h is constant, l is the distance between columns, the moment of inertia of a column is constant throughout one storey, and the moments of inertia of the floor beams throughout the building are the same.

If I_{cr} is the moment of inertia of the columns of the rth storey and I_b the

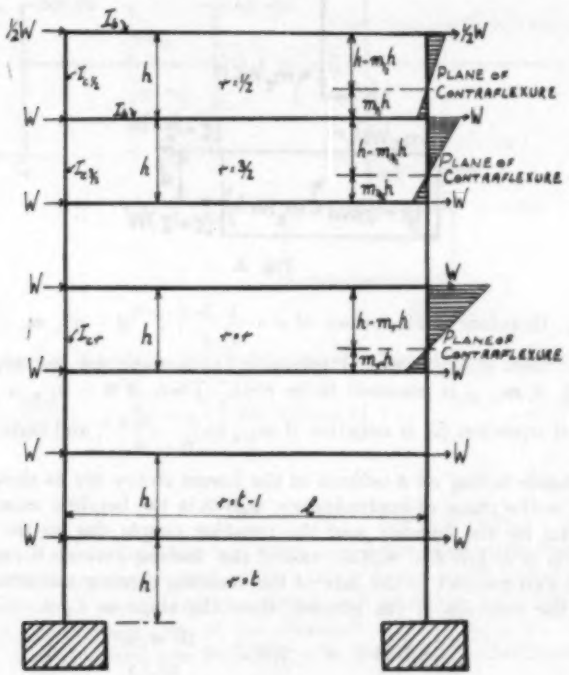


Fig. 1.

moment of inertia of the beams, the ratio n_r of the stiffness of the columns to the stiffness of the beams is $\frac{II_{cr}}{hI_b}$. The height of the plane of contraflexure from the floor of the rth storey is $m_r h$. For storeys other than the top and bottom storeys it has been shown[1] that

$$m_r = \frac{m_{(r+1)}n_r(r+1) + m_{(r-1)}n_r(r-1) - n_r + 3r}{r(2n_r + 6)} \quad . \quad (1)$$

From this expression,

$$m_{(r+1)} - m_r = \frac{n_r(r-1)(m_r - m_{(r-1)}) + 6rm_r - 3r + n_r}{n_r(r+1)} \quad . \quad (2)$$

If $m_r < m_{(r-1)}$ and $r \neq \frac{1}{2}$, the first term of the numerator on the right-hand side of this equation is negative. The whole numerator is negative if also $6rm_r - 3r + n_r < 0$, that is, if $m_r < \frac{1}{2} - \frac{n_r}{6r}$. If the numerator is negative

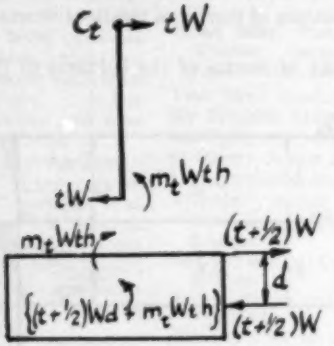


Fig. 2.

$m_{(r+1)} < m_r$; therefore if for values of $r = \frac{3}{2}, \frac{5}{2}, \ldots (t-1)$, $m_r < \frac{1}{2} - \frac{n_r}{6r}$ and $m_r < m_{(r-1)}$, then $m_{(r+1)} < m_r$. Expression (1) is valid for the top storey, for which $r = \frac{1}{2}$, if $m_{(r-1)}$ is assumed to be zero. Then, if $6 - n_{1/2}$ is positive, the numerator of equation (2) is negative if $m_{1/2} < \frac{3 - 2n_{1/2}}{6 - n_{1/2}}$, and then $m_{3/2} < m_{1/2}$.

If the forces acting on a column of the lowest storey are as shown in *Fig.* 2 in which C_t is the plane of contraflexure, m_tWth is the bending moment applied to the footing by the column, and the resisting couple due to the reaction of the ground is $(t + \frac{1}{2})Wd + m_tWth$, and if the footing rotates θ radians about a horizontal axis parallel to the face of the building causing a restraining couple $K\theta$ due to the reaction of the ground, then the slope at C_t is

$$\frac{1}{K}[m_tWth + (t + \tfrac{1}{2})Wd] + \frac{tWm_t^2h^2}{2EI_{ct}} \quad . \quad . \quad . \quad . \quad (3)$$

1.—A. L. L. Baker. "Reinforced Concrete." Concrete Publications, Ltd.

From consideration of the floor above (*Fig.* 3), the slope at C_t is

$$\frac{tW(1-m_t)^2h^2}{2EI_{ct}} + \frac{lh}{6EI_b}[(t-1)Wm_{(t-1)} + tW(1-m_t)].$$

Equating these expressions for the slope at C_t, if $q = \frac{6EI_{ct}}{Kh}$,

$$m_t(6t + tn_t + qt) = m_{(t-1)}[tn_t - n_t] + tn_t + 3t - q(t + \tfrac{1}{2})\frac{d}{h} \quad . \quad (4)$$

whence

$$n_t(t-1)(m_t - m_{(t-1)}) + \left[m_t(6t + qt + n_t) - tn_t - 3t + q(t + \tfrac{1}{2})\frac{d}{h}\right] = 0.$$

If $m_t < m_{(t-1)}$ the first term is negative and the term in square brackets is positive;

therefore

$$m_t > \frac{tn_t + 3t - q(t + \tfrac{1}{2})\frac{d}{h}}{6t + qt + n_t}.$$

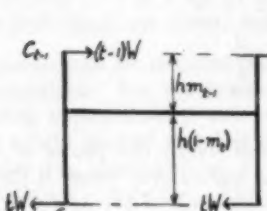


Fig. 3.

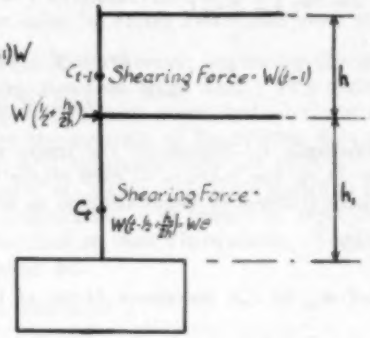


Fig. 4.

Therefore no value of m_r can be less than the least of the following expressions.

$$\frac{3 - 2n_{1/2}}{6 - n_{1/2}}, \qquad \left(\frac{1}{2} - \frac{n_r}{6r}\right) \text{ for values of } r = \frac{3}{2}, \frac{5}{2} \ldots (t-1);$$

$$\text{and} \quad \frac{tn_t + 3t - q(t + \tfrac{1}{2})\frac{d}{h}}{6t + qt + n_t} \quad . \quad . \quad . \quad (5)$$

It can be shown in a like way that m_r cannot be greater than the greatest values of the same expressions. The theory is not valid unless the planes of contraflexure are " real ", and the easiest way to ensure that this is the case is to arrange the cross-section of the members so that $n_r < 3r$. This is not a necessary condition but one for which the planes of contraflexure are always real.

If the positions of the planes of contraflexure in storeys p and s are known, for intermediate storeys the limiting values of m_r are respectively the greatest and least values of the following expressions.

$$m_p;\ \left(\frac{1}{2}-\frac{n_r}{6r}\right) \text{ for values of } r=(p+1),\ (p+2),\ \ldots\ (s-1);\ \text{and } m_s.$$

If the height of the plane of contraflexure above the floor of storey p is between $y_p h$ and $z_p h$, and in storey s is between $y_s h$ and $z_s h$, then the limiting values of m_r are the greatest and least values of the following expressions.

$$y_p;\ z_p;\ \left(\frac{1}{2}-\frac{n_r}{6r}\right) \text{ for values of } r=(p+1),\ (p+2),\ \ldots\ (s-1);\ y_s;\ \text{and } z_s.$$

If the limits between which the plane of contraflexure lies are known only for storey p, then the expressions for the limits in storeys above storey p are

$$\frac{3-2n_{1/2}}{6-n_{1/2}};\ \left(\frac{1}{2}-\frac{n_r}{6r}\right) \text{ for values of } r=\frac{3}{2},\ \frac{5}{2},\ \ldots\ (p-1);\ y_s;\ \text{and } z_s,$$

and for storeys below storey p are

$$y_p;\ z_p;\ \left(\frac{1}{2}-\frac{n_r}{6r}\right) \text{ for values of } r=(p+1),\ (p+2)\ \ldots\ (t-1);$$

$$\text{and } \frac{tn_t+3t-q(t+\frac{1}{2})\frac{d}{h}}{6t+qt+nt}.$$

Example 1.—Determine the limits between which the planes of contraflexure lie in the columns of each storey of an eight-storey frame if the ratios of column stiffness to beam stiffness n_r are as shown in *Table* 1, $q=5{\cdot}55$ and $\frac{d}{h}=0{\cdot}25$.

Let m_{rl} be the successive terms of the expressions (5). Then, if $r=\frac{1}{2}$,

$$m_{rl}=\frac{3-2\times 0{\cdot}3}{6-0{\cdot}3}=0{\cdot}4211;\ \text{if } r=\frac{3}{2},\ m_{rl}=\frac{1}{2}-\frac{0{\cdot}8}{9}=4{\cdot}112.$$

Other values are in *Table* 1. The limits are such that the planes of contraflexure lie between 0·4334 and 0·4112 of the storey-height. The fully computed values are also given in the table for comparison.

TABLE 1.—EXAMPLE 1.

r	n_r	m_{rl} from expressions (5)	m_r fully computed
$\frac{1}{2}$	0·3	0·4211	0·4198
$\frac{3}{2}$	0·8	0·4112	0·4120
$\frac{5}{2}$	1·3	0·4133	0·4144
$\frac{7}{2}$	1·7	0·4190	0·4191
$\frac{9}{2}$	2·1	0·4223	0·4217
$\frac{11}{2}$	2·6	0·4212	0·4221
$\frac{13}{2}$	3·1	0·4205	0·4240
$\frac{15}{2}$	3·7	0·4334	0·4313

The limits of the position of the plane of contraflexure are not always as close as those in example 1, but by the use of expressions (1) and (4) more accurate values of the lower limit M_r min. can be determined for each storey as in the following example.

Example 2.—Determine the limits between which lie the planes of contraflexure in the columns of each storey of an eight-storey frame, if the ratios of column stiffness to beam stiffness n_r are as shown in *Table 2*, $q = 22{\cdot}2$ and $\frac{d}{h} = 0{\cdot}25$.

The greatest and least values of m_r are derived from expressions (5) as in Example 1, and are 0·1250 and 0·3937, which are not close enough for the accuracy required. Improved values for the lower limits are required since the greatest bending moments result from planes of contraflexure at the lower limit.

The value of $m_{1/2}$ is calculated from equation (1), using the lowest possible values of $m_{(r-1)}$ and $m_{(r+1)}$, zero and 0·1250 respectively. The value of $m_{1/2}$ is 0·1250, which is no improvement. Calculating $m_{3/2}$ from equation (1) with the lowest possible values of $m_{1/2}$ and $m_{5/2}$, that is 0·1250, the lowest value of $m_{3/2}$ is 0·1344, and this is the lowest possible value for storey $r = \frac{3}{2}$ and lower storeys. This process can be repeated for all the lower storeys, improving the original approximations. For the lowest storey equation (4) is used. This method of improving approximations can be repeated several times, but accuracy should not be sought beyond that justified by the accuracy of the original data and in particular the assessment of q. The first approximations are generally found to be sufficiently accurate. The values of m_{r1} for example 2 are shown in *Table 2* with the fully computed values for comparison. The results of improving the approximation twice by the method described are shown in brackets. The limiting values for a particular storey are called M_r.

TABLE 2.—EXAMPLE 2.

r	n_r	m_{r1} from expressions (5)	M_r max.	M_r min. (*improved*)	m_r fully computed
$\frac{1}{2}$	1·2	0·1250	0·3937	0·1250 (0·1290)	0·1379
$\frac{3}{2}$	3·2	0·1444	0·3937	0·1344 (0·1418)	0·1552
$\frac{5}{2}$	5·2	0·1533	0·3937	0·1413	0·1703
$\frac{7}{2}$	6·8	0·1762	0·3937	0·1520	0·1908
$\frac{9}{2}$	8·4	0·1889	0·3937	0·1617	0·2123
$\frac{11}{2}$	10·4	0·1848	0·3937	0·1669	0·2393
$\frac{13}{2}$	12·4	0·1821	0·3937	0·1699 (0·2456)	0·2848
$\frac{15}{2}$	14·8	0·3937	0·3937	0·3270	0·3607

If the height of the lowest storey is not equal to the height of the other storeys it is shown in the appendix that the term $\dfrac{tn_t + 3t - q(t + \frac{1}{2})\frac{d}{h}}{6t + qt + n_t}$ in expressions (5)

If the positions of the planes of contraflexure in storeys p and s are known, for intermediate storeys the limiting values of m_r are respectively the greatest and least values of the following expressions.

m_p; $\left(\frac{1}{2} - \frac{n_r}{6r}\right)$ for values of $r = (p + 1), (p + 2), \ldots (s - 1)$; and m_s.

If the height of the plane of contraflexure above the floor of storey p is between $y_p h$ and $z_p h$, and in storey s is between $y_s h$ and $z_s h$, then the limiting values of m_r are the greatest and least values of the following expressions.

y_p; z_p; $\left(\frac{1}{2} - \frac{n_r}{6r}\right)$ for values of $r = (p + 1), (p + 2), \ldots (s - 1)$; y_s; and z_s.

If the limits between which the plane of contraflexure lies are known only for storey p, then the expressions for the limits in storeys above storey p are

$\frac{3 - 2n_{1/2}}{6 - n_{1/2}}$; $\left(\frac{1}{2} - \frac{n_r}{6r}\right)$ for values of $r = \frac{3}{2}, \frac{5}{2}, \ldots (p - 1)$; y_s; and z_s,

and for storeys below storey p are

$$y_p;\ z_p;\ \left(\frac{1}{2} - \frac{n_r}{6r}\right) \text{ for values of } r = (p + 1), (p + 2) \ldots (t - 1);$$

$$\text{and } \frac{tn_t + 3t - q(t + \frac{1}{2})\frac{d}{h}}{6t + qt + nt}.$$

Example 1.—Determine the limits between which the planes of contraflexure lie in the columns of each storey of an eight-storey frame if the ratios of column stiffness to beam stiffness n_r are as shown in *Table* 1, $q = 5{\cdot}55$ and $\frac{d}{h} = 0{\cdot}25$.

Let m_{r1} be the successive terms of the expressions (5). Then, if $r = \frac{1}{2}$,

$m_{r1} = \frac{3 - 2 \times 0{\cdot}3}{6 - 0{\cdot}3} = 0{\cdot}4211$; if $r = \frac{3}{2}$, $m_{r1} = \frac{1}{2} - \frac{0{\cdot}8}{9} = 4{\cdot}112$.

Other values are in *Table* 1. The limits are such that the planes of contraflexure lie between 0·4334 and 0·4112 of the storey-height. The fully computed values are also given in the table for comparison.

TABLE 1.—EXAMPLE 1.

r	n_r	m_{r1} from expressions (5)	m_r fully computed
$\frac{1}{2}$	0·3	0·4211	0·4198
$\frac{3}{2}$	0·8	0·4112	0·4120
$\frac{5}{2}$	1·3	0·4133	0·4144
$\frac{7}{2}$	1·7	0·4190	0·4191
$\frac{9}{2}$	2·1	0·4223	0·4217
$\frac{11}{2}$	2·6	0·4212	0·4221
$\frac{13}{2}$	3·1	0·4205	0·4240
$\frac{15}{2}$	3·7	0·4334	0·4313

The limits of the position of the plane of contraflexure are not always as close as those in example 1, but by the use of expressions (1) and (4) more accurate values of the lower limit M_r min. can be determined for each storey as in the following example.

Example 2.—Determine the limits between which lie the planes of contraflexure in the columns of each storey of an eight-storey frame, if the ratios of column stiffness to beam stiffness n_r are as shown in *Table 2*, $q = 22{\cdot}2$ and $\frac{d}{h} = 0{\cdot}25$.

The greatest and least values of m_r are derived from expressions (5) as in Example 1, and are 0·1250 and 0·3937, which are not close enough for the accuracy required. Improved values for the lower limits are required since the greatest bending moments result from planes of contraflexure at the lower limit.

The value of $m_{1/2}$ is calculated from equation (1), using the lowest possible values of $m_{(r-1)}$ and $m_{(r+1)}$, zero and 0·1250 respectively. The value of $m_{1/2}$ is 0·1250, which is no improvement. Calculating $m_{3/2}$ from equation (1) with the lowest possible values of $m_{1/2}$ and $m_{5/2}$, that is 0·1250, the lowest value of $m_{3/2}$ is 0·1344, and this is the lowest possible value for storey $r = \frac{3}{2}$ and lower storeys. This process can be repeated for all the lower storeys, improving the original approximations. For the lowest storey equation (4) is used. This method of improving approximations can be repeated several times, but accuracy should not be sought beyond that justified by the accuracy of the original data and in particular the assessment of q. The first approximations are generally found to be sufficiently accurate. The values of m_{r1} for example 2 are shown in *Table 2* with the fully computed values for comparison. The results of improving the approximation twice by the method described are shown in brackets. The limiting values for a particular storey are called M_r.

TABLE 2.—EXAMPLE 2.

r	n_r	m_{r1} from expressions (5)	M_r max.	M_r min. (*improved*)	m_r fully computed
$\frac{1}{2}$	1·2	0·1250	0·3937	0·1250 (0·1290)	0·1379
$\frac{3}{2}$	3·2	0·1444	0·3937	0·1344 (0·1418)	0·1552
$\frac{5}{2}$	5·2	0·1533	0·3937	0·1413	0·1703
$\frac{7}{2}$	6·8	0·1762	0·3937	0·1520	0·1908
$\frac{9}{2}$	8·4	0·1889	0·3937	0·1617	0·2123
$\frac{11}{2}$	10·4	0·1848	0·3937	0·1669	0·2393
$\frac{13}{2}$	12·4	0·1821	0·3937	0·1699 (0·2456)	0·2848
$\frac{15}{2}$	14·8	0·3937	0·3937	0·3270	0·3607

If the height of the lowest storey is not equal to the height of the other storeys it is shown in the appendix that the term $\dfrac{tn_t + 3t - q(t + \frac{1}{2})\frac{d}{h}}{6t + qt + n_t}$ in expressions (5)

should be replaced by

$$\frac{qt + 6xt + tn_t - q\theta x + 3\theta x^2 - q\left(\theta + \frac{x}{2}\right)\frac{d}{h}}{qt + 6xt + n_t} \quad . \quad . \quad (5a)$$

in which $\theta = t - \frac{1}{2} + \frac{h_1}{2h}$, $x = \frac{h_1}{h}$, $\frac{6EI_{ct}}{Kh} = q$, and $n_t = \frac{lI_{ct}}{hI_b}$. The method may then be applied as before. If it is desired to improve the limits for the lowest storey, equation (8) in the appendix should be applied, and for the storey in which $r = t - 1$ the appropriate equation is expression (7) given in the appendix. The following example illustrates the use of this method for frames in which the height of the lowest storey differs from that of the other storeys.

Example 3.—Determine the limits between which lie the planes of contraflexure in each storey of an eight-storey frame. The lowest storey is $1\frac{2}{3}$ the height of the other storeys which are of equal height h. The horizontal reaction of the ground due to wind acts at a distance $\frac{h}{3}$ below the top of the foundation. The value of q is 3·083, and $n_t = 6{\cdot}167$. The following values apply, $\frac{d}{h} = \frac{1}{3}$, $t = \frac{15}{2}$ and $x = \frac{h_1}{h} = \frac{5}{3}$. The values of r, n_r, and the calculated values of m_{rl} are shown in *Table* 3, with the fully computed values of m_r for comparison.

TABLE 3.—EXAMPLE 3.

r	n_r	m_{rl} from expressions (5) and (5a)	M_r max.	M_r min.	m_r fully computed
$\frac{1}{8}$	0·3	0·4211			0·4198
$\frac{2}{8}$	0·8	0·4112			0·4120
$\frac{3}{8}$	1·3	0·4133			0·4144
$\frac{4}{8}$	1·7	0·4190	0·4223*	0·2871*	0·4187
$\frac{5}{8}$	2·1	0·4223			0·4200
$\frac{6}{8}$	2·6	0·4212			0·4152
$\frac{7}{8}$	3·1	0·4205			0·4000
$\frac{8}{8}$	6·167	0·2871	0·5904	0·6681	0·6126

* The limits 0·4223 and 0·2871 apply to m_t'. Expression (6) is used to relate these values to limits of m_t.

APPENDIX.

HEIGHT OF LOWEST STOREY DIFFERING FROM HEIGHT OF OTHER STOREYS.

Let the height of the lowest storey be h_1 and the height of its plane of contraflexure C_t be $m_t h_1$, then the shearing forces at C_t are as shown in *Fig.* 4.

The equation relating $m_{(t-1)}$ to m_t and $m_{(t-2)}$ is obtained by equating the slopes of the column above and below $c_{(t-1)}$. If, for convenience, $t - \frac{1}{2} + \frac{h_1}{2h} = \theta$ and $\frac{h_1}{h} = x$, then

$$m_{(t-1)} = \frac{m_{(t-2)}n_{(t-1)}(t-2) + m_t n_{(t-1)}x\theta - n_{(t-1)}x\theta + n_{(t-1)}t - n_{(t-1)} + 3(t-1)}{2(t-1)(n_{(t-1)} + 3)}$$

If
$$m_t' = \frac{x\theta}{f}(m_t - 1) + 1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

$$m_{(t-1)} = \frac{m_{(t-2)}n_{(t-1)}(t-2) + m_t'n_{(t-1)}^2 - n_{(t-1)} + 3(t-1)}{2(t-1)(n_{(t-1)} + 3)} \quad . \quad . \quad (7)$$

which has a similar form to expression (1).

The expression m_t' should be used in place of m_t in the analysis of frames in which the height of the lowest storey differs from the rest and the limits determined for m_t' should be related to m_t using expression (5a).

To determine the equation relating $m_{(t-1)}$ to m_t' expressions for the slope of the column above and below C_t are equated as follows:

$$\frac{1}{K}\left[m_th_1W\theta + \left(\theta + \frac{h_1}{2h}\right)Wd\right] + \frac{W\theta m_t^2h_1^2}{2EI_{ct}} = \frac{W\theta(1-m_t)^2h_1^2}{2EI_{ct}} + \frac{l}{6EI_b}[h(t-1)Wm_{(t-1)} + W\theta h_1(1-m_t)].$$

If $\frac{h_1}{h} = x$, $\frac{6EI_{ct}}{Kh} = q$, and noting that $n_t = \frac{lI_{ct}}{hI_b}$,

$$m_t = \frac{m_{(t-1)}(t-1)n_t + 3\theta x^2 + n_t\theta x - q\left(\theta + \frac{x}{2}\right)\frac{d}{h}}{q\theta x + 6\theta x^2 + n_t\theta x},$$

or
$$m_t' = \frac{m_{(t-1)}(t-1)n_t - 3\theta x^2 - q\left(\theta + \frac{x}{2}\right)\frac{d}{h} + qt + 6xt + tn_t - q\theta x}{t(q + 6x + n_t)} \quad . \quad . \quad (8)$$

Therefore,

$$(m_t' - m_{(t-1)})n_t(t-1) + \left[m_t'(qt + 6xt + n_t) + 3\theta x^2 + q\left(\theta + \frac{x}{2}\right)\frac{d}{h} - qt - 6xt - tn_t + q\theta x\right] = 0.$$

If $m_t' < m_{(t-1)}$, the first term in this expression is negative and hence the term in square brackets is positive, that is,

$$m_r > \frac{qt + 6xt + tn_t - q\theta x - 3\theta x^2 - q\left(\theta + \frac{x}{2}\right)\frac{d}{h}}{qt + 6xt + n_t} \quad . \quad . \quad . \quad (9)$$

Therefore the term $\frac{tn_t + 3t - q(t + \frac{1}{2})\frac{d}{h}}{6t + qt + n_t}$ in expression (5) should be replaced by expression (9) when determining the limits for m_r and improving these limits, if the height of the lowest storey differs from the rest. The limits, for the storey in which $r = t$, refer to m_t', and equation (8) is the appropriate one for improving the limits in the lowest storey and equation (7) for the storey in which $r = t - 1$.

Technical Writing.

THE City and Guilds Institute are continuing their courses, which were inaugurated in 1958, on the subjects of Technical Authorship and Technical Illustration. The course on authorship is in two parts, namely at ordinary and advanced levels. Copies of the syllabus and list of recommended books for both courses, which are held at various centres throughout England, are obtainable from the City and Guilds Institute, 76 Portland Place, London, W.1 (Price 1s.).

The Professional Engineer.

A ONE-DAY conference on "The Professional Engineer—His Employment and Development" has been organised by the Engineers' Guild Ltd., to be held at the Connaught Rooms, Great Queen Street, London, W.C.2, on Wednesday, March 22. The Chairman will be Sir Hugh Beaver, K.B.E. The fee is four guineas. Particulars are obtainable from the Secretary, Engineers' Guild Ltd., 201 High Holborn, London, W.C.1.

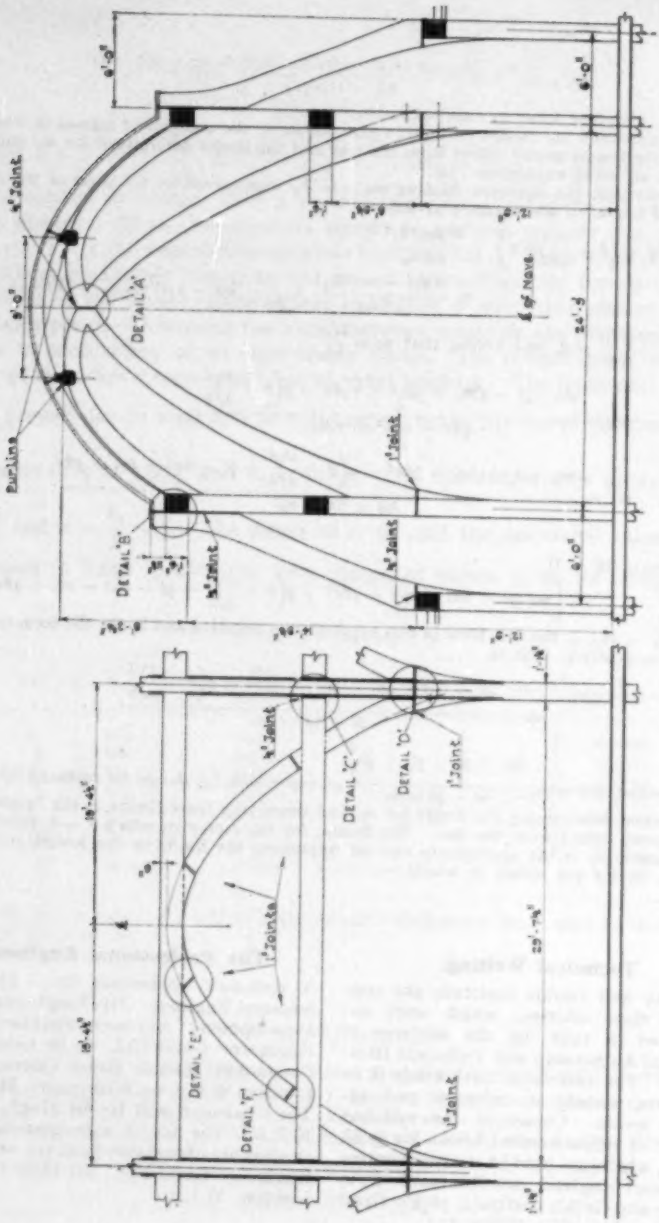


Fig. 2.
(See facing page.)

Church and Campanile at Stevenage.

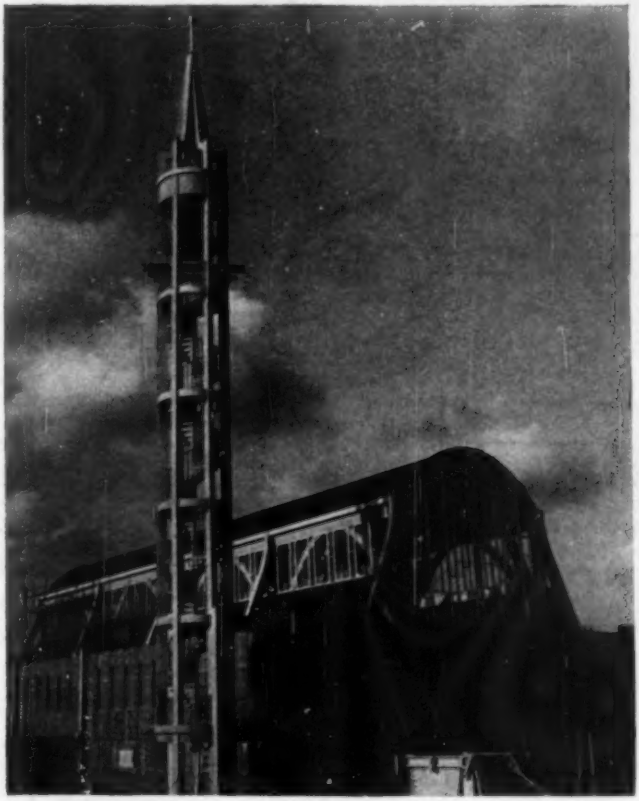


Fig. 1.

THE structure of the main building of the new church (*Fig.* 1) of St. George at Stevenage New Town comprises five pairs of transverse interlacing precast reinforced concrete parabolic arches at 30-ft. 9-in. centres (*Fig.* 4). The heaviest precast member weighs 6 tons. Part of each arch projects externally to produce an effect similar to a flying buttress. Other single arches are provided longitudinally. The connections between the precast members were made with mortar. The building is 124 ft. long, 60 ft. wide and 47 ft. high, and the finish in part is of flints embedded in precast concrete panels. The arches inside the building are visible from outside owing to the wide expanse of fenestration. The crypt, Lady chapel, chapter room and offices are on the ground floor. The porch, baptistry and the main body of the church are on the first floor.

The first-floor slab is 10 in. thick, is of hollow-tile construction, spans in two directions and is supported mainly on brick walls. Precast steps outside the building lead to the main entrance. The roof over the porch and baptistry is of curved precast units, spanning about 12 ft., with exposed-aggregate soffits.

The main transverse frames of the church and the two longitudinal frames were precast in parts as indicated in

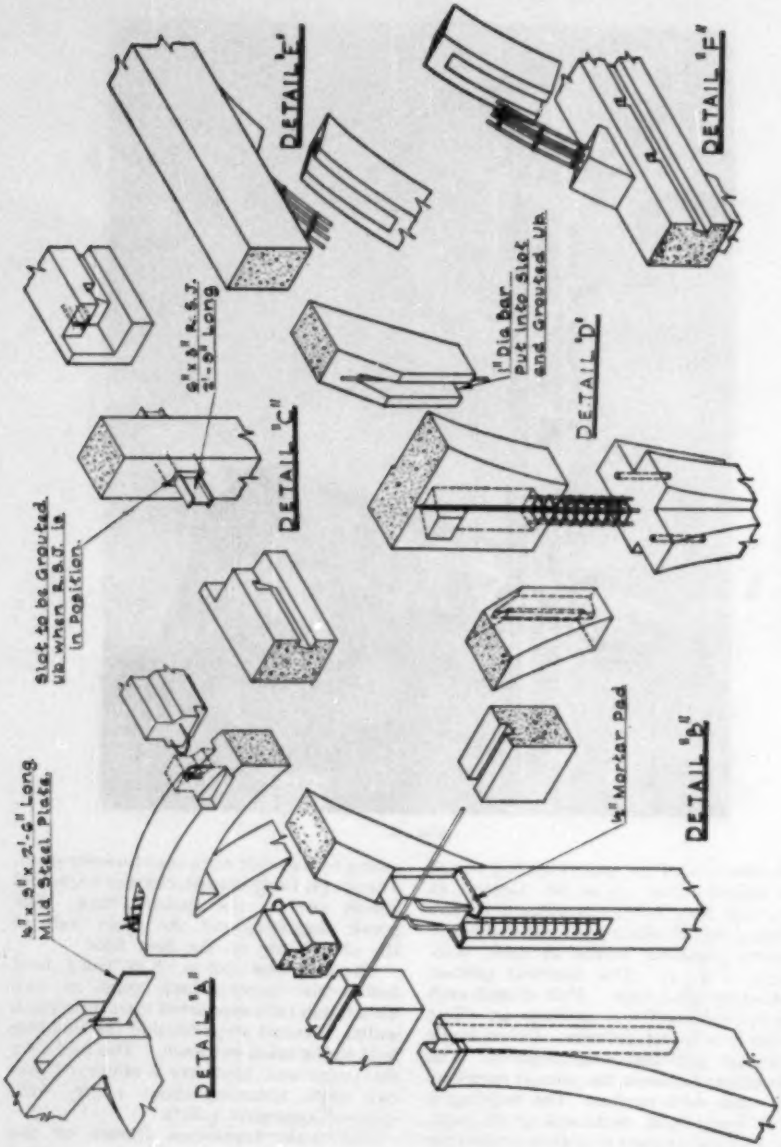

DETAIL 'E'
DETAIL 'E'
1" Dia Bar Put into Slot and Grouted Up.
DETAIL 'D'
6"x4" R.S.J. 2'-9" Long
DETAIL 'C'
Slot to be Grouted up when R.S.J. is in Position.
½" Mortar Pad
DETAIL 'B'
½"x 8"x 2'-6" Long Mild Steel Plate.
DETAIL 'A'

Fig. 3.

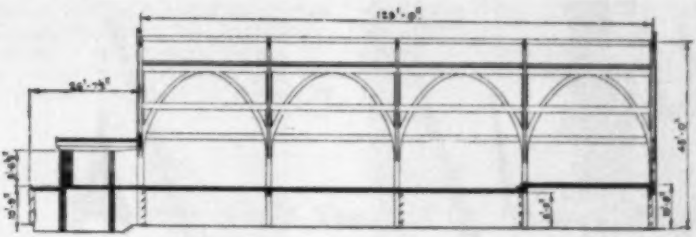


Fig. 4.

Fig. 2. Two precast purlins 15 in. deep carried on the transverse frames support the roof which is of timber and perforated hardboard. The arches in the longitudinal frames and the duplication of the transverse arches are for æsthetic purposes only.

Full-size templates were made for the complicated precast units. The templates were cut at the positions of the joints and the moulds made to fit the templates. The main columns were cast in lengths of about 23 ft. and grouted into pockets 12 in. deep in the caps of the foundation piles. Holes were provided in some of the columns for steel connectors to support the first floor and bars projected from the top of the columns into pockets in the double-arch units. Concrete was placed around these bars through a hole on one side of each arch-unit. These and other details of connections are shown in *Fig. 3*.

Adjacent to the church is a campanile 106 ft. high and of 9-ft. 6-in. diameter. The prestressed concrete columns, which are 15 in. by 9 in. in cross-section, were precast each in one length of 81 ft. The ring-beams connecting the columns at 13-ft. 9-in. centres were cast insitu. The pinnacle of the campanile is 20 ft. high and is constructed of six precast members which were assembled on the ground (*Fig. 5*) and lifted into position. Erection was carried out with a derrick crane having a 120-jib (*Fig. 6*).

Two platforms (*Fig. 1*) connect the church and the campanile and provide lateral stiffening. A helical stair in the

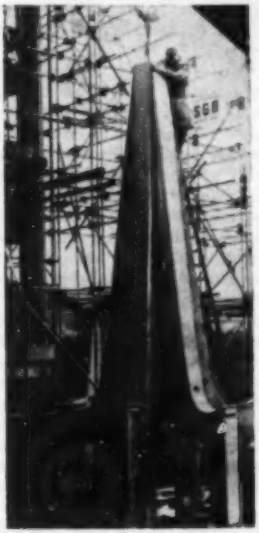


Fig. 5.—Pinnacle Assembled on the Ground.

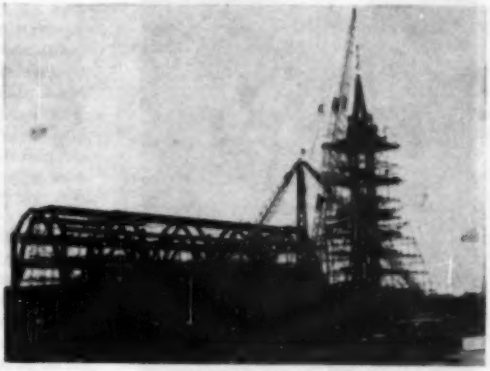


Fig. 6.

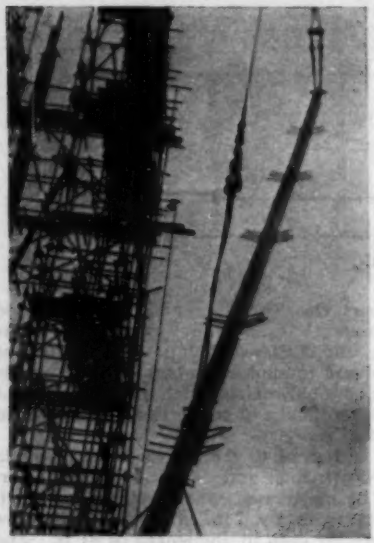


Fig. 7.—Erection of Columns for Campanile.

Fig. 8.—Placing Columns in Position.

Fig. 9.—Erecting the Pinnacle.

campanile is supported by a central column 9 in. in diameter. The stair is of precast steps, every third step being connected to the prestressed columns of the campanile. At the top of these columns a cast-insitu ring-beam 3 ft. 9 in. deep and a platform 15 in. thick stiffen the columns and support electronic bell equipment.

The erection of the 81-ft. columns required the attachment of a second hook to the jib-crane and a second independently-controlled rope-drum. Each column was first raised horizontally by two hooks, one at the top of the column and one 14 ft. 6 in. from the bottom. The columns, while suspended, were brought to a vertical position by raising the upper hook and dropping the lower one (*Fig.* 7). The lower hook was then removed and the column lowered into a pocket 4 ft. deep in the piled foundation. The column was supported in tubular-steel scaffolding, on three sides of the campanile (*Fig.* 8), while it was grouted in position. The erection of each column required about half an hour. The pinnacle, which weighs 7 tons, was raised (*Fig.* 9) by means of a steel eye fixed in a steel tube in the apex of the pinnacle which now supports a cross.

To prevent damage to the long precast columns during transportation wooden boxes with rubber pads were provided at

points of support. The columns were transported at night to avoid obstructing roads during the day. The foundations are eighty-five 50-ton bored piles.

The cost of the structural work was about £35,000, the cost of the entire scheme being about £120,000. The architect is Lord Mottistone of Messrs. Seely & Paget, and the consulting engineers are Messrs. Hajnal & Myers. The contractors were Messrs. Rattee & Kett, Ltd.

A Prestressed Test-bed for Engines.

THE extension of the works of Messrs. Burmeister & Wain Co., Ltd., of Copenhagen, includes a new steel-frame building which is used for assembling and testing marine engines. The design and construction of the prestressed concrete foundation for the engines are described by Mr. J. Schmidt in "Nordisk Betong" for March 1960, from which the following description is abstracted.

The foundation is 84 m. (275 ft. 7 in.) long and 7 m. (23 ft. 9 in.) wide and is large enough for three of the largest marine engines that have so far been made to be submitted to test runs simultaneously. The foundation was constructed in three separate longitudinal parts so as to reduce the loss of prestress due to friction. These parts were stressed together by means of cables in combination with flat-jacks placed in the joints (*Fig.* 1).

There are forty-four Freyssinet cables C, and eight Freyssi flat-jacks B arranged as shown in *Fig.* 1. There are also five concrete wedges A covered with sheet steel. D indicates two pipes through which oil is supplied to the testing machines mounted on the foundations.

The flat-jacks could be expanded up to 25 mm. (1 in.) and thereby offer resistance to the prestressing forces. By maintaining the manometric pressure of the flat-jacks constant during the period between partial and final prestressing, the jacks were enabled to follow the movements of the joints and to ensure the transference of prestressing forces across the joints. The concrete wedges were designed to act in place of the flat-jacks if these should unexpectedly fail.

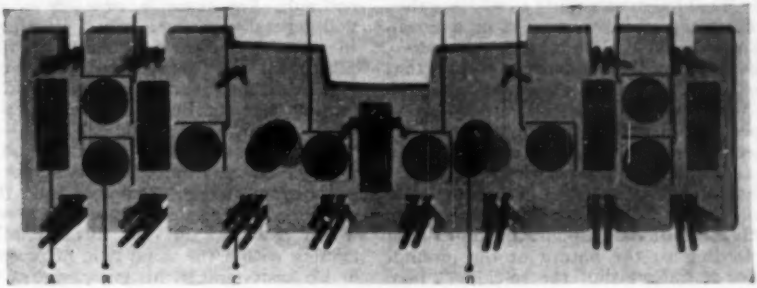


Fig. 1.

After filling the joints with mortar, the three parts of the foundation became an integral unit. Due to the low rotational speeds (110 to 125 r.p.m.) of the engines, the foundation was designed to have a high frequency instead of a low frequency which is commonly preferable. The consequence of this is that the natural frequency of the combined machine and foundation exceeds considerably the frequency of the machine alone. This should exclude all danger of resonance, and it should therefore be possible to regard the transference of forces from the foundation to the base as being predominantly static and not dynamic.

Resistance of Concrete Piles to Penetration.

WITH reference to the article in our number for November, 1960, entitled "The Resistance of Concrete Piles to Penetration" by Donovan H. Lee, MR. F. R. BULLEN writes as follows.

"No doubt Mr. Lee is correct when he uses this title, since it would seem from what he says that he is seeking to measure or calculate the resistance to penetration during driving. Whilst this may be desirable information, I cannot but feel that the real object of a piling formula should be to calculate the bearing capacity of a pile under a static load and the two results are not necessarily equal.

Since my paper* has been quoted several times by Mr. Lee, I think I ought to make it clear that my approach to the problem was to calculate the bearing capacity of the toe and then to add or subtract the frictional forces according to whether they constituted positive support or a down-drag. In these circumstances, therefore, Mr. Lee is hardly correct when he says that my formula does not allow for 'take up'.

My conception of the pile-driving operation is that during driving the pile meets relatively little side resistance because of the flexure of the pile, but when the driving has ceased the ground closes around the pile and thereby creates either additional support or down-drag according to the nature of the ground. Thus it follows that the so-called 'take up' is appropriately allowed for in the value of the friction used. Similarly, by applying a plasticity factor, I have sought to build into the formula an allowance for the relaxation in toe-value which occurs in materials of a plastic or viscous nature. Such materials are usually clays, that is cohesive materials, but a similar phenomenon is encountered in the case of fine sands which in other contexts might be described as 'quick'.

Finally, Mr. Donovan Lee's treatment is a useful way of dealing with the problem, but care must be taken to ensure that the real quantity required, namely the bearing capacity at the toe under a static load, is not confused with the resistance to penetration during driving."

MR. DONOVAN H. LEE replies.—"I hope it was clear that the work of Mr. E. A. L. Smith in America and my own graphical method deal solely with resistance to penetration at the time of driving. It should not be overlooked that Mr. Hiley and other writers have been at pains to make it clear that their particular formulæ or methods deal only with the resistance during driving and leave to the engineer the responsibility of estimating the corresponding ultimate static load or the safe load for any particular acceptable settlement.

Some formulæ go beyond this and, with additional factors (for example, to allow for 'take up' or for the effects of a steady load in a cohesive soil increasing settlement by gradual expulsion of water), go beyond the calculation of the resistance to penetration at the time of driving.

The danger with these formulæ is that the conditions assumed may not be realised. For example, one pile in clay will generally produce only a fraction of the time-settlement to which a group of piles will be subjected. The writer regrets if the article could be read to suggest that Mr. Bullen did not allow for 'take up' in his formula.

Whether an engineer uses a method or formula to determine the resistance to penetration during driving and then computes the 'take up', or the reverse where possible, and the effect of other loading within the same bulb of pressure in a cohesive soil by his knowledge of soil mechanics, or whether he utilises a formula like that of Mr. Bullen and makes further adjustments for the effects of groups of piles is for him to choose. It seems to the writer, however, that recent methods of obtaining the resistance to penetration can be of great assistance if used in conjunction with present knowledge of soil mechanics, and this procedure is possibly better than utilising one of the recent formulæ that go beyond that as they may tempt engineers not to appreciate the need to make corrections.

Unfortunately, many serious mistakes in interpreting piling formulæ in the past, and even to-day, arise from confusion between the resistance to penetration during driving and the ultimate static load. Notwithstanding that many writers have pointed this out, their formulæ have been misused and said to be incorrect when the formula was correct within the limits claimed by its author."

* "A New Calculation for the Bearing Capacity of Driven Piles." Proc. Inst. Civil Eng., May 1959.

The Design of Eccentrically-loaded Circular Columns by the Load-factor Method.—I.

By J. D. BENNETT, B.Eng.

THE charts for the design of circular columns by the load-factor method, which are given in this article, are prepared by a method in which all the assumptions are in accordance with the recommendations of the British Standard Code of Practice No. 114 (1957). The charts are complementary to those for the design of rectangular columns subjected to combined bending and axial load which were given in this journal for November and December 1957, March and May 1958, and later in a book.*

The charts are simple to use to determine the most economical reinforcement in a circular column. An approximate design of a square column subjected to bending in two directions can be obtained by assuming an equivalent circular column and distributing the reinforcement equally near the four sides of the square column.

NOTATION.—As far as possible, the notation is the same as that used in the Code of Practice, but it is necessary to use some additional symbols in the charts and in the derivation of the basic formulæ.

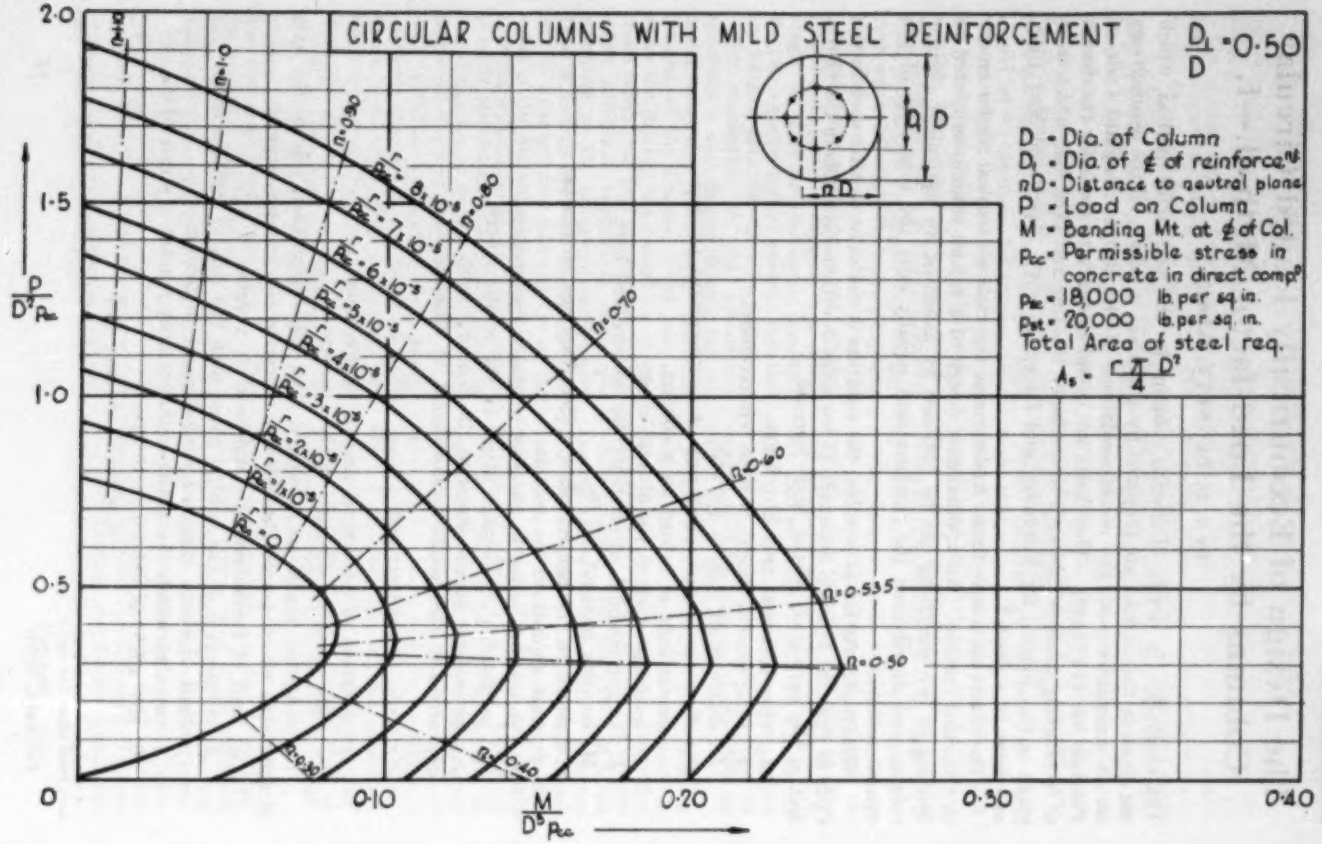
A_c: cross-sectional area of column.
A_s: total cross-sectional area of reinforcement.
D: diameter of column ($= 2r_0$).
D_1: diameter of equivalent ring of reinforcement ($= 2r_1$).
e: eccentricity of a load on a column.
E: modulus of elasticity of mild steel.
E_s: secant modulus of elasticity for cold-worked bars.
M: bending moment ($= Pe$).
nD: depth to the neutral plane. (See diagrams on charts.)
P: load applied on the column.
p_{cc}: permissible stress in concrete in direct compression.
p_{sc}: permissible compressive stress in the reinforcement.
p_{st}: permissible tensile stress in the reinforcement.
$2p_t$: stress in the tensile reinforcement at ultimate load.
r: ratio of reinforcement $\left(= \frac{A_s}{A_c} = \frac{4D_1t}{D^2}\right)$.
t: thickness of equivalent ring of reinforcement.
u: cube crushing strength of the concrete.

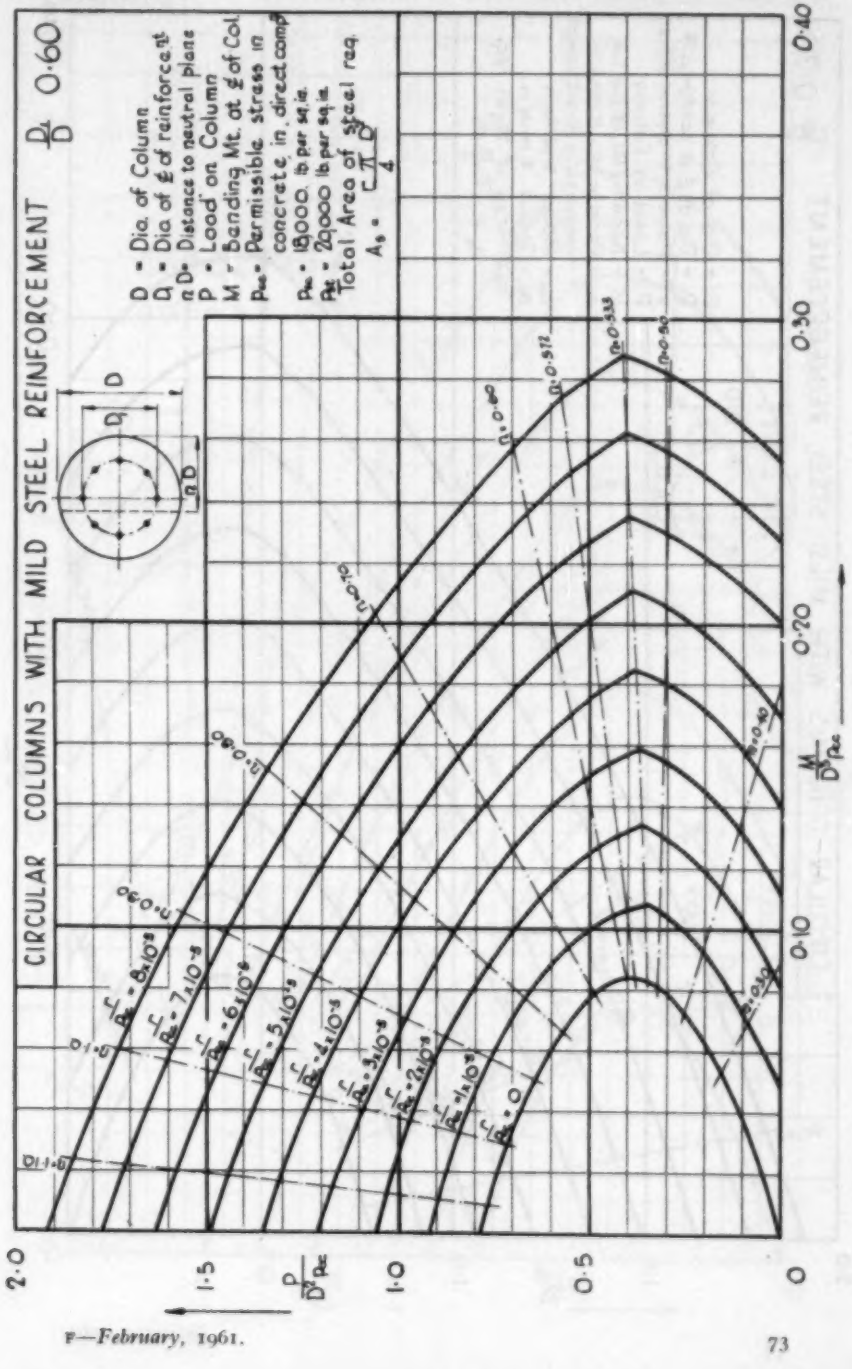
The following angles are measured from the compression side of the axis of symmetry which passes through the centre of the column section.

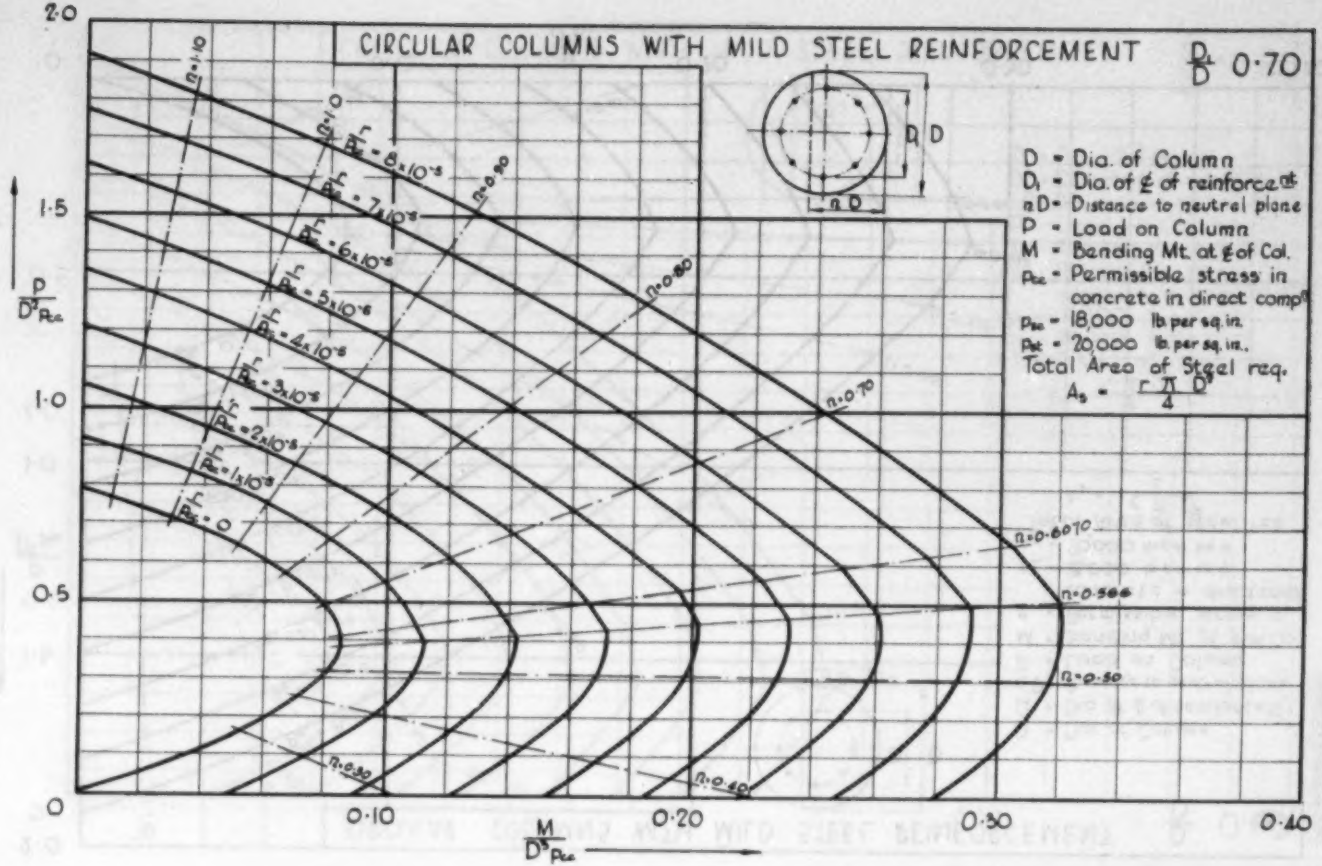
α: limit of reinforcement compressed at a stress of p_{sc}.
β: intersection of the neutral plane with the ring of reinforcement.
γ: limit of elastic range of reinforcement in tension.
δ: point at which the tensile reinforcement reaches a stress of p_{st}.
θ: any angle as defined in the text.

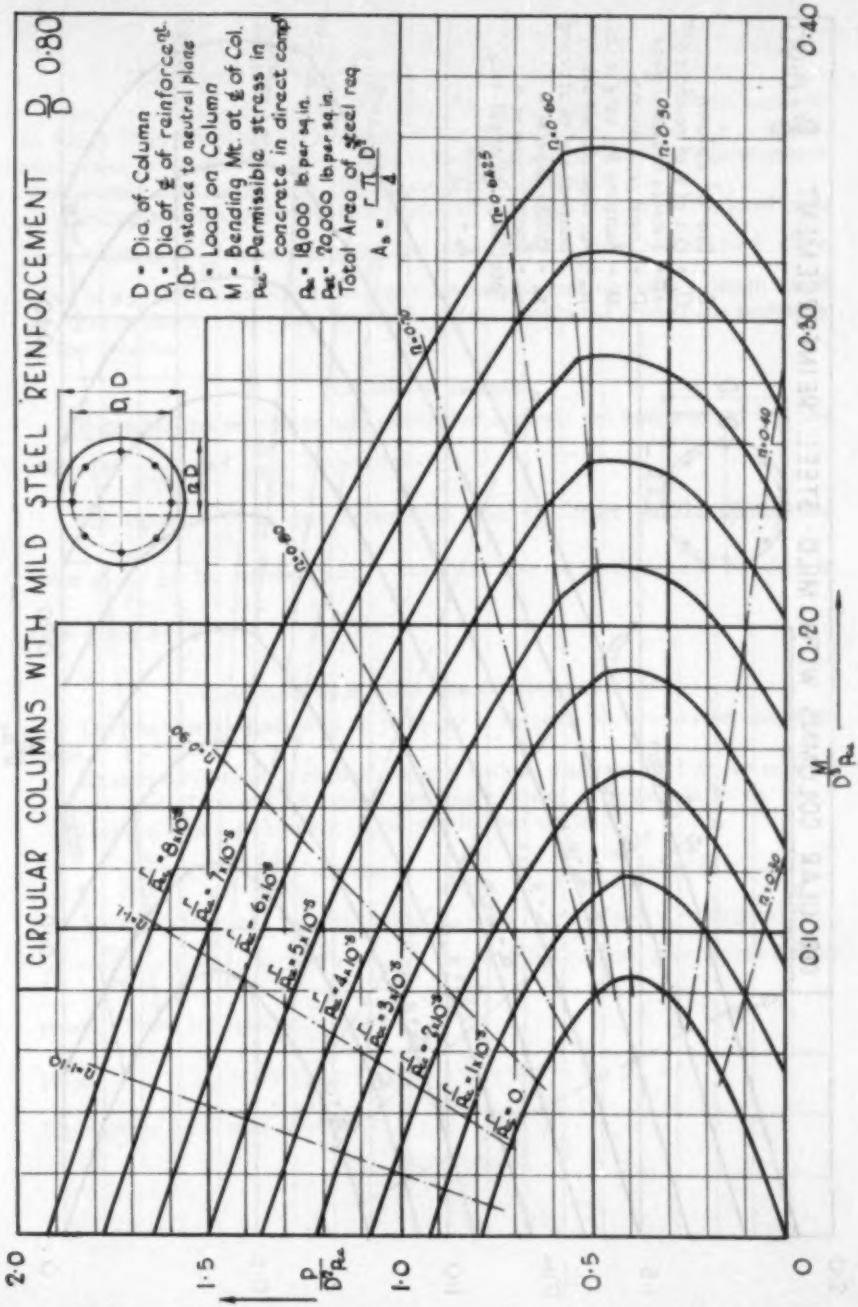
(*Continued on page 77.*)

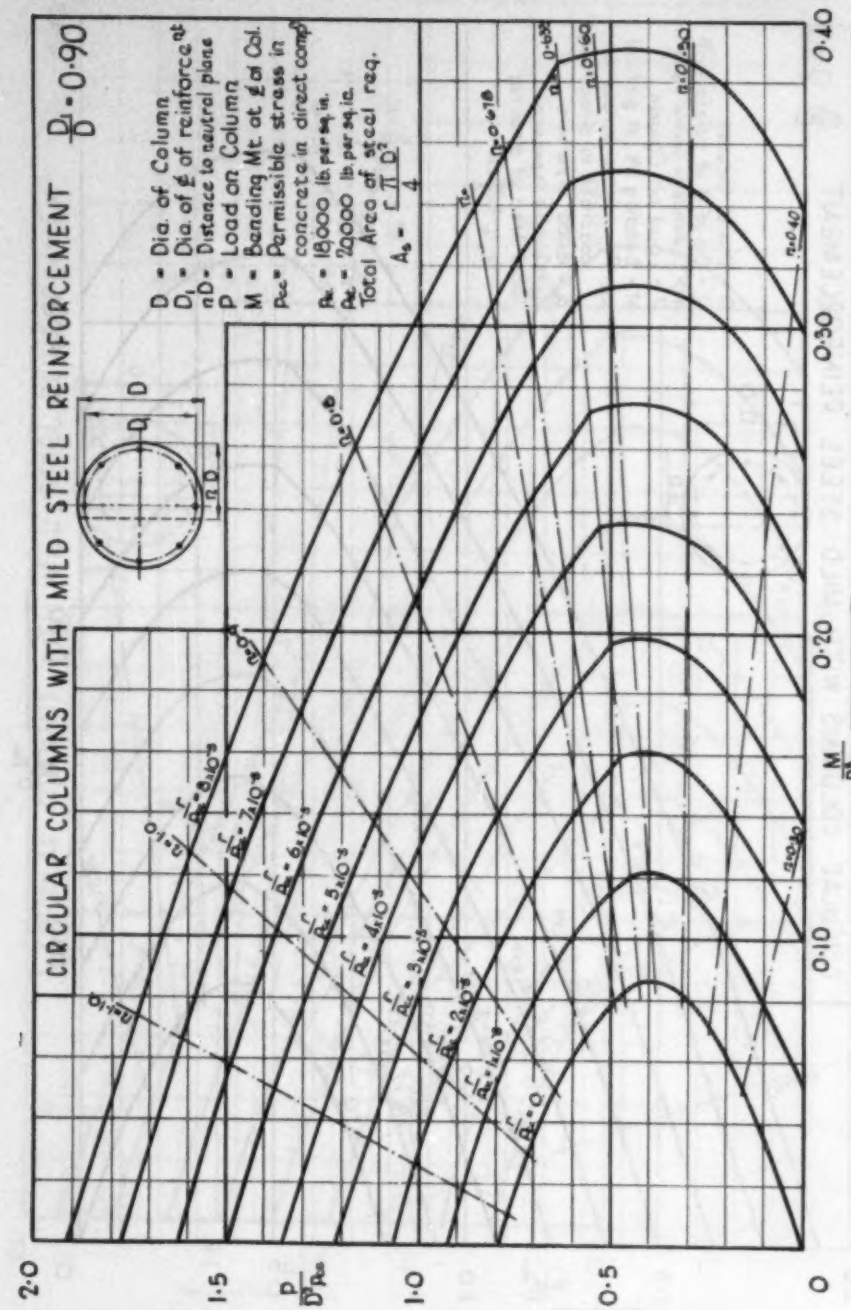
* "Design of Eccentrically-loaded Columns by the Load-factor Method". By J. D. Bennett. (Published by Concrete Publications, Ltd. 1958.)


CIRCULAR COLUMNS WITH MILD STEEL REINFORCEMENT
D1/D = 0·50
D = Dia. of Column
D1 = Dia. of ℄ of reinforcement
nD = Distance to neutral plane
P = Load on Column
M = Bending Mt. at ℄ of Col.
Pcc = Permissible stress in concrete in direct compn
Pcc = 1,800 lb. per sq. in.
Pst = 20,000 lb. per sq. in.
Total Area of steel req.
As = pπ/4 D²
P/D³Pcc
M/D³Pcc
n=0·90
n=0·80
n=0·70
n=0·60
n=0·535
n=0·50
n=0·40
p=0·10
p=0·05
p=0
0
0·10
0·20
0·30
0·40
0·5
1·0
1·5
2·0


CIRCULAR COLUMNS WITH MILD STEEL REINFORCEMENT
d/D 0·60
D = Dia. of Column
Q = Dia. of ℄ of reinforcement
nD = Distance to neutral plane
P = Load on Column
M = Bending Mt. at ℄ of Col.
Pac = Permissible stress in concrete in direct comp.
Pac = 1500 lb per sq in.
Pac = 2000 lb per sq in.
Total Area of Steel req. A_s = (π/4) D²
n=0·40
n=0·30
n=0·20
n=0·10
n=0·533
n=0·512
n=0·480
n=0·430
n=0·350
n=0·250
n=0·0
f/Pac = 8·10⁻³
f/Pac = 7·10⁻³
f/Pac = 6·10⁻³
f/Pac = 5·10⁻³
f/Pac = 4·10⁻³
f/Pac = 3·10⁻³
f/Pac = 2·10⁻³
f/Pac = 1·10⁻³
f/Pac = 0
D/t = 6
D/t = 15
0·40
0·30
0·20
0·10
M/D³Pac
2·0
1·5
1·0
0·5
0
P/D²Pac


CIRCULAR COLUMNS WITH MILD STEEL REINFORCEMENT
D1/D 0·70
D = Dia. of Column
D1 = Dia. of ℄ of reinforcement
nD = Distance to neutral plane
P = Load on Column
M = Bending Mt. at ℄ of Col.
fcc = Permissible stress in concrete in direct comp.
fcc = 18,000 lb per sq. in.
fst = 20,000 lb per sq. in.
Total Area of Steel req.
As = r π/4 D²
P/D²fcc
M/D³fcc
2·0
1·5
1·0
0·5
0
0·10
0·20
0·30
0·40
n=1·0
n=0·90
n=0·80
n=0·70
n=0·60
n=0·50
n=0·40
n=0·30
r/mc = 8×10⁻⁵
r/mc = 7×10⁻⁵
r/mc = 6×10⁻⁵
r/mc = 5×10⁻⁵
r/mc = 4×10⁻⁵
r/mc = 3×10⁻⁵
r/mc = 2×10⁻⁵
r/mc = 1×10⁻⁵
r/mc = 0
nD
D


CIRCULAR COLUMNS WITH MILD STEEL REINFORCEMENT D/D 0·80
D - Dia of Column
D1 - Dia of ℄ of reinforcement
nD - Distance to neutral plane
P - Load on Column
M - Bending Mt. at ℄ of Col.
Pac - Permissible stress in concrete in direct comp
Pac = 18,000 lb per sq in.
Pac = 20,000 lb per sq in.
Total Area of steel req
As = (r π D²)/4
n=0·60
n=0·50
n=0·40
n=0·30
n=0·25
0·40
0·30
0·20
0·10
M/D³Pac
P/D²Pac
2·0
1·5
1·0
0·5
0


CIRCULAR COLUMNS WITH MILD STEEL REINFORCEMENT D1/D = 0·90
D = Dia. of Column
D1 = Dia. of ℄ of reinforcement
nD = Distance to neutral plane
P = Load on Column
M = Bending Mt. at ℄ of Col.
Pcc = Permissible stress in concrete in direct comp.
fc = 18000 lb per sq. in.
fst = 24000 lb per sq. in.
Total Area of steel req. As = p π D²/4
P/D²Pcc
M/D³Pcc
2·0
1·5
1·0
0·5
0
0·10
0·20
0·30
0·40

(*continued from page 71.*)

ϕ: intersection of the neutral plane with the outside of the column.
ϕ_1: limit of the concrete stress-block on the outside of the column.

The following assumptions are made: the maximum stress in the concrete at failure does not exceed two-thirds of the prism strength, that is $\frac{2}{3} \times 0{\cdot}76u$; the column is designed to have a load-factor generally of two; the reinforcement is uniformly distributed in a ring of mean diameter D_1.

The charts, which are for columns reinforced with mild steel or cold-worked bars, are for values of $\frac{D_1}{D}$ from 0·5 to 0·9 and for values of $\frac{r}{p_{cc}}$ up to 8×10^{-5}. The value of n is also indicated on the charts and can be used to verify a design based on other methods if the reinforcement is widely distributed around the perimeter of the column.

Method of Design.

A suitable size of column and mixture of concrete are first selected and the values of $\frac{P}{D^2 p_{cc}}$ and $\frac{M}{D^3 p_{cc}}$ determined.

The value of $\frac{r}{p_{cc}}$ is then determined from the chart for the appropriate value of $\frac{D_1}{D}$ (or by interpolation). The total area of reinforcement required is then given by $A_s = \frac{r}{p_{cc}} \times p_{cc} \frac{\pi}{4} D^2$.

Columns with Mild Steel Reinforcement.

The five charts on pages 72 to 76 apply to columns with mild steel reinforcement.

EXAMPLE.—Design a circular column with a diameter of 2 ft. to support a direct load of 300,000 lb. and a bending moment of 1,500,000 lb.-in.

Using a 1 : 2 : 4 concrete; $p_{cc} = 760$ lb. per sq. in.

$$\frac{P}{D^2 p_{cc}} = \frac{300{,}000}{24^2 \times 760} = 0{\cdot}685. \qquad \frac{M}{D^3 p_{cc}} = \frac{1{,}500{,}000}{24^3 \times 760} = 0{\cdot}143.$$

Using 1½-in. cover to ¼-in. links, $D_1 = 24 - 3 - 0{\cdot}5 - 1 = 19{\cdot}5$ in. $\frac{D_1}{D} = \frac{19{\cdot}5}{24} = 0{\cdot}81$. The graph for $\frac{D_1}{D} = 0{\cdot}80$ will therefore give a conservative result. From this graph $\frac{r}{p_{cc}} = 2{\cdot}8 \times 10^{-5}$.

Hence $$A_s = 2{\cdot}8 \times 10^{-5} \times 760 \times \frac{\pi}{4} \times 24^2 = 9{\cdot}6 \text{ sq. in.}$$

Use sixteen ⅞-in. mild steel bars.

(*To be continued.*)

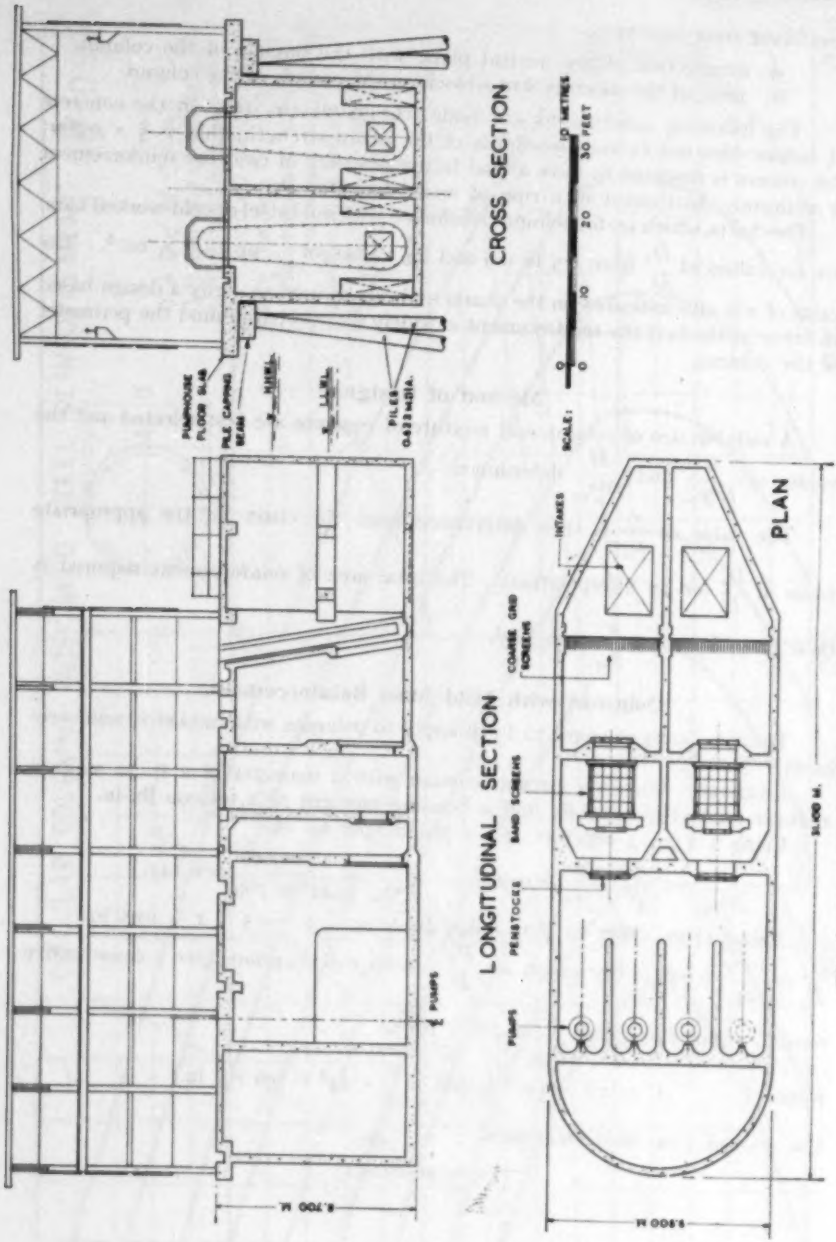


Fig. 2.—General Arrangement of the Intake Pumphouse and Chamber.
(*See facing page.*)

Intake Works at Basrah Power Station.

THE new power station constructed for the Iraq Government to supply electricity for Southern Iraq is on the right bank of the Qarmat Ali river at Basrah and has a total capacity of 45 MW.; provision is made for future extension. The station is of conventional design but the design and construction of the cooling-water intake is of interest. The quantity of water required for the present and future installations is 17,000 cu. m. per hour (247,000 cu. ft. per hour) at full load. At the site of the power station the river is about 300 m. (984 ft.) wide, with steep banks and a depth of water of about 16 m. (52 ft. 6 in.). The river is tidal, with an average range of 1·5 m. (4 ft. 11 in.). The velocity of the current is about 0·69 m. per second (1·31 knots) at the ebb and only slightly less when flowing. Considerable erosion of the river banks has occurred in the past during floods but the extensive flood prevention works carried out recently in Northern Iraq have greatly reduced these floods and therefore the rate of erosion of the river bank.

Duplicate sets of self-cleaning course grid screens, fine band screens and screen-washing pumps are provided in the intake chamber for protection from debris from the marsh lands and a reed-processing plant upstream of the intake. Either set of band screens may be isolated for inspection and servicing by means of vertical-lift penstocks. The main pump suction chamber can similarly be isolated and emptied. Three vertical pumps are installed and there is provision for a fourth. Other equipment in the pumphouse is a pump to supply the fire main, a water-level indicator in the main pump chamber, and a 10-tonne hand-operated overhead crane in the superstructure.

Fig. 1.—Chamber in position ready for concreting Floor Slab.

Design.

The location of the pumphouse relative to the river bank, and its general form, were determined by the following requirements. It had to be constructed in water sufficiently deep at all times to provide the requisite head of water at the pump intakes. The chamber intakes had to be as far as possible from supporting piles where reeds and debris might collect. The design had to allow for the construction of the concrete chamber out of the water as the excessive depth of water made the use of a cofferdam undesirable.

The intake chamber, approximately 31 m. by 10 m. by 8 m. high (101 ft. 9 in. by 32 ft. 10 in. by 26 ft. 3 in.), is suspended between two rows of piles, each row having twelve piles with a capping beam. The upstream end of the chamber is cylindrical (*Fig.* 2) and the downstream end narrows in width, so as to minimise the obstruction to the passage of debris. The main intakes to the chamber are at the underside of the downstream part. A reinforced concrete access-way supporting a road 3·5 m. (11 ft. 6 in.) wide, a circulating water pipe, a drainage outfall pipe and a fire main extends out to the pumphouse.

The pumphouse and access-way are supported on vertical and raking piles. There is a 10-mm. (0·4-in.) gap between the access-way and intake structure at the edge of the intake pile-cap beam.

Piling.

The piles are of a type sometimes known as "Gambia" piles. Each pile comprises

a steel tube of 0·622-m. (2-ft. 0½-in.) outside diameter and 0·010 m. (0·4 in). thick, with a cast steel driving-shoe. The piles were driven by a 5-tonne hammer acting on a reinforced concrete plug in the bottom of each tube.

The vertical and raking piles were driven by means of a special piling-frame mounted on two pontoons. After pitching, the pile was guided down the leaders through strong collars, the lower collars being so designed that a new length of tube could be clamped to the driven tube to facilitate butt-welding. The piles for the intake chamber reached the required set after penetrating between 6·5 m. and 7·8 m. (21 ft. 4 in. and 25 ft. 6 in.) into the river bed with fairly hard driving.

After driving the tube a cage of reinforcement comprising six 1¼-in. bars was lowered into each tube which was then filled with concrete. The concrete was fed into the tubes from a skip with bottom-opening conical flap-doors designed for the purpose, and suspended from the piling frame. The pile-cap beams were then concreted.

The design load on each pile is 80 tons which can be supported by the reinforced concrete core alone so that the steel tube can be totally corroded without endangering the structure. The raking piles resist the horizontal forces which arise from the thrust on the piles and intake chamber due to river currents and wind on the superstructure.

Fig. 3.—Supporting Cradle hanging from Capping Beams.

Construction.

Construction of the intake as a caisson was not adopted because of possible difficulties in manœuvring a floating caisson safely on to the supporting beams, and because of the disadvantage of providing a concreting plant at a slipway at some distance from the power station. The scheme adopted was to cast the intake chamber on a temporary frame hanging above its final position and then jack the chamber down to its position in the water.

The intake was constructed as follows. A heavy supporting cradle was assembled and suspended above water level from the pile-cap beams. The intake chamber was concreted up to the underside of the floor, with sealed water intakes and lowered by means of jacks to capping-beam level. The floor slab was cast and after maturing supported the hanging weight of the chamber. The temporary supporting frame was removed, the superstructure constructed and the crane and all equipment was installed.

The cradle (*Fig.* 3) was formed of double 24-in. by 7½-in. transverse steel I-beams with 8·5-in. by 0·86-in. flange plates, and 12-in. by 5-in. longitudinal steel I-beams at 1·175-m. (3-ft. 10-in.) centres. There were twelve transverse beams along the length of the chamber base, each suspended from hangers of square box-section passing between the piles through holes in the pile-cap beam. The weight of the intake chamber, which was about 1500 tons, was borne entirely by the suspended cradle. When the cradle was in position, timber shuttering for the base of the chamber was laid on it.

In order to facilitate recovery of the cradle after use, wires attached each member to two buoys, one at each end of the structure.

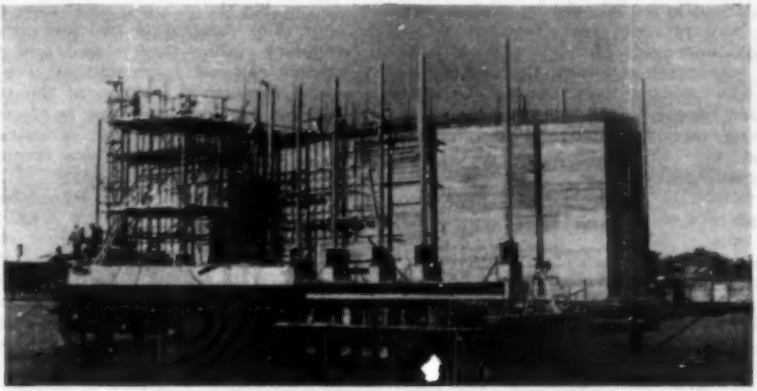


Fig. 4.—Intake Chamber ready for lowering.

Concreting.

Concrete was mixed in two batching plants on the site of the power station. Concrete for the intake was brought along the completed access-way by lorry, delivered into a hopper whence it was taken in large barrows to the shuttering. A hoist was provided to assist in concreting the walls. All concrete is similar and has not less than 385 kg. of cement per cubic metre (650 lb. per cubic yard). A coarse aggregate of ¾-in. maximum size was used and was obtained from gravel deposits to the south of Basrah and washed, screened, and graded on site. Naturally occurring fine aggregate was used. Test cubes were made to ascertain the proportions of coarse and fine aggregates giving the greatest strength, and the chosen proportions were used in weigh-batching all materials. With a water-cement ratio of 0·47, the strengths of cubes made throughout the concreting operations were about 5000 lb. per square inch.

The base slab, which contains 115 cu. m. (150 cu. yd.) of concrete, was cast in one operation. The construction joints have rubber water-bars. The walls were concreted in two lifts each of 3·75 m. (12 ft. 4 in.); the concrete being deposited 85 cu. m. (111 cu. yd.) at a time in six operations. This concrete and that for the base slab contains a retarder in the proportion of 0·5 per cent. of the weight of cement.

Concreting occupied a period of three months.

Lowering the Intake Chamber.

Preparations were made for lowering the intake chamber while concreting was in progress. Each of the twenty-four hangers was supported on the capping beam by means of specially designed mild steel brackets and cotters. The brackets rested on hardwood packing on concrete blocks (*Fig.* 3). Two hydraulic jacks each of 50-tonne capacity were fitted to each bracket. The hydraulic pumps and control panel were set up on a staging at the downstream end of the chamber. A high-pressure pump and a low-pressure pump were installed, with a supply system

so arranged that the master valve controlled all jacks simultaneously, whilst four subsidiary valves could be used to adjust the pressure on any of four groups of six pairs of jacks in case uneven lowering took place. Each pair of jacks could also be isolated in case of failure; pressure gauges were fitted to all lines.

On completion of the concreting (*Fig.* 4) the chamber was made ready for lowering. The main water intakes were sealed, so that the chamber could later be emptied for the installation of pumps and screens, and two temporary valves were opened at the downstream end to allow water to enter during lowering. Scaffolding was erected to support the floor slab shuttering, and the structure balanced longitudinally by adding water to an upstream compartment until all jacks were raised through equal distances in momentarily raising the structure.

The lowering operation was as follows. Each hanger had two cotters, the weight being carried initially on the lower of the two. Under the action of the low-pressure pump the jacks raised the secondary bracket to the higher cotter. The high-pressure pump was then connected to the jacks by means of the master control and the weight transferred from the lower cotter. The lower cotter was then removed and the whole structure allowed to descend under the control of the master valve on the high-pressure system, until it came to rest on the lower cotter which had meanwhile been transferred to the adjoining hole in the hanger. This operation was repeated until the chamber was in its final position (*Fig.* 1). It was found that wind-blown particles could reach the sealing glands of the jacks causing minor oil leaks. When this happened, the affected jack was quickly replaced, by one of two spare jacks, and immediately cleaned. The structure was lowered in three days requiring about twenty-two hours operating time. When the chamber had been lowered the floor slab of the pumphouse was concreted. The cantilevered nibs of this slab, when resting on the pile-cap beams, were to support the full weight of the completed intake structure, and therefore the supporting cradle and hangers were not dismantled until the slab had attained its full strength.

To enable the screens, penstocks, and pumps to be installed the two downstream temporary valves were closed and the chamber emptied by means of a 4-in. pump. Meanwhile erection of the steel superstructure and 10-tonne hand-crane continued. Installation of mechanical plant followed. The intake was commissioned during November 1959.

The consulting engineers are Ewbank & Partners, Ltd., who act on behalf of the Government of Iraq, to whom acknowledgment is made for permission to present this article. The consulting engineers for the civil engineering works are Messrs. G. Maunsell & Partners, and the architects are Messrs. Wilson, Mason & Partners. The civil engineering contractors are Messrs. Polensky & Zöllner of Frankfurt-on-Main.

New Jetty at Milford Haven.

In the number of this journal for January 1961, a description is given of the construction of a new jetty for the Esso Petroleum Co., Ltd. In the following are given further particulars of this work.

Precast Concrete Road Slabs.

The precast concrete slabs for the road, which are each 35 ft. long and 8 ft. wide, were made in the same casting yard as the piles. They were cast in concrete moulds. The surfaces of the moulds were painted with a mixture of whitewash and glue size and, when this had dried, the reinforcement and concrete were placed in the mould. When the slab had hardened, it was raised from the mould by hydraulic pressure. Water was pumped into pipes embedded in, and opening into, the bottom of the mould. Steel plates covering the openings of the pipes were contained in recesses in the bottom of the mould so that the soffits of the slabs were smooth. A water pressure of about 10 lb. per square inch beneath the plates was sufficient to raise the precast slabs from the mould. The slabs were then removed by mobile cranes.

Hollow Piles.

The method of forming the cavity in the large hollow cylindrical prestressed concrete piles is patented by Dow-Mac (Products), Ltd.

The jetty was designed and constructed by Messrs. John Mowlem Co., Ltd.

Improvements on the Great North Road.*

II.—SOUTHERN SECTION.—LONDON TO NOTTINGHAMSHIRE.

Rowley Lane Bridge.

Widening of the Barnet by-pass section of the Great North Road is proceeding and involves the construction of an overbridge (*Fig.* 1) at Rowley Lane. The bridge, which is designed for standard Ministry of Transport loading, has two main spans each of 53 ft. 3 in. and two smaller side spans. The side spans were chosen in preference to abutment walls to give a more open appearance. The deck of the main spans is carried on prestressed precast beams, with pretensioned steel, at 5 ft. centres acting in conjunction with a 7-in. cast-insitu slab. The beams are of Y-trough section, with which shape the dead weight and the transverse span of the deck are reduced. The beams have considerable stiffness and each is designed to carry half the load of a lane of traffic without transference of load to adjacent beams. The permanent shuttering of the deck slab is of 1½-in. precast planks spanning about 2 ft. For the erection of the precast beams, the main road was closed to traffic on one day from 7.30 a.m. to 4 p.m., during which period fourteen concrete beams, each weighing over 11 tons, were lifted and lowered into place by a 50-ton crane (*Fig.* 2).

Each side span is designed as a portal frame, virtually pin-jointed at the pier. A thick cast-insitu deck-slab acts with the walls to resist the lateral loads due to pressure and traffic. Rocking slabs at each end of the bridge rest on the filling and are provided to mitigate the effects of settlement of the embankments.

Fig. 1.—Rowley Lane Bridge.

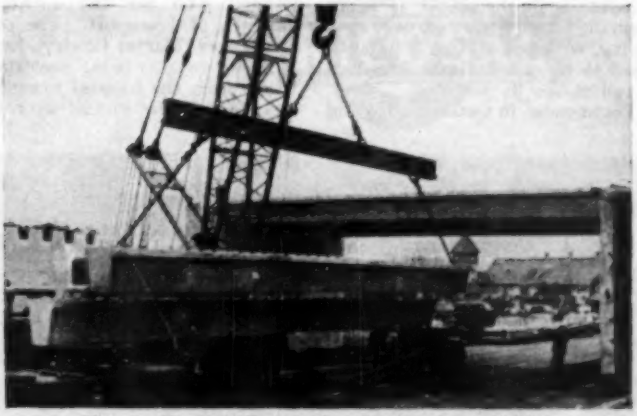


Fig. 2.—Precast Beams for Rowley Lane Bridge.

* Continued from January, 1961, in which number the works in Yorkshire are described

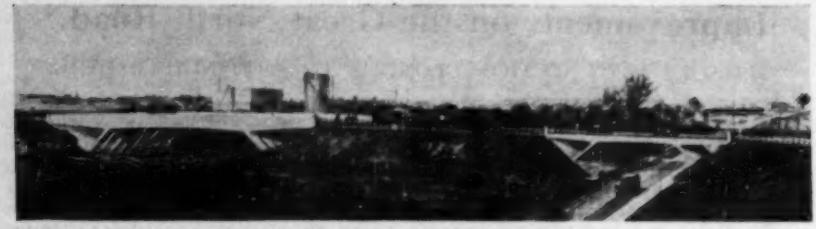


Fig. 3.—Bridges at Stevenage.

The simple strip foundations are at a depth of about 7 ft. 6 in. and are on stiff brown clay. The greatest bearing pressure is 1¼ tons per square foot. To reduce the size of the foundations of the abutment walls, balancing beams are provided between the foundations of the outer and inner walls.

Allowance is made for thermal movements between the main and side spans, by dividing the top 9 ft. of the supporting walls into two flexible leaves, with 1 in. of compressible material between the leaves. All horizontal loads on the main spans are carried by the central wall where the continuity of the deck slab is severed to prevent overstressing the main beams due to compression in the bottom flange. Waterstops are provided at all permanent joints in the deck slab. The deck is waterproofed by hot-bonded copper-lined bituminous sheeting. The carriageway which is 24 ft. wide is finished with 3½ in. of rolled asphalt. The galvanised mild steel hand-railing, with box-section standards, is designed to resist a horizontal load on the top rail of 200 lb. per foot.

Work commenced in October 1959 and was completed in May 1960. The consulting engineers were Messrs. Leslie Turner and Partners. Supervision of the works, the cost of which, including 4½ miles of road, will be about £500,000, is being carried out for the Ministry of Transport by H. S. Andrews, M.I.C.E., County Surveyor of Middlesex. The contractors are Messrs. John Laing & Son, Ltd.

Stevenage By-pass.

Work was commenced recently on a motorway 7½ miles long to by-pass Stevenage and nearby villages, and it is proposed to extend this road northwards for 4½ miles to by-pass Baldock. There will be thirteen bridges over or under the new road, two subways for cyclists and pedestrians, and three subways for pedestrians only. The designers of the motorway, the cost of which will be about £1,800,000, are the Hertfordshire County Council who are the agents for the Ministry of Transport. The contractors are Messrs. Martin Cowley, Ltd.

A new road is being constructed eastwards from the by-pass towards Stevenage and will be carried over the Great

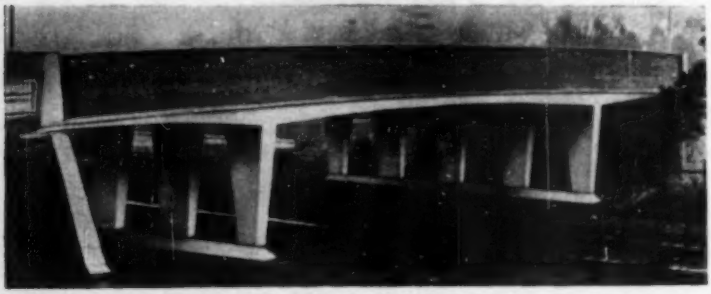


Fig. 4.—Six Hills Bridge, Stevenage.

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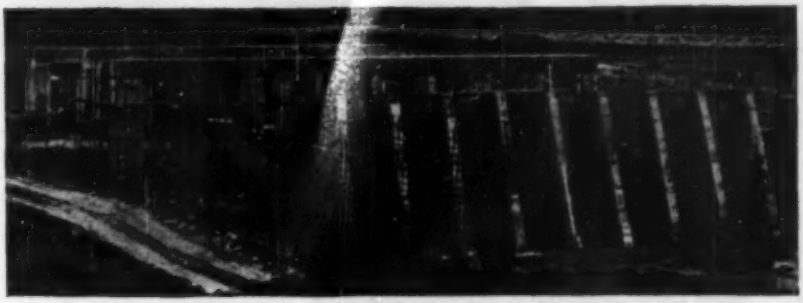


Fig. 5.—Bridge on the Stamford By-pass.

North Road and the adjacent parallel railway by the two bridges in the reproduction of a perspective drawing in (*Fig.* 3). The bridge in (*Fig.* 4) is the Six Hills bridge, near Stevenage, which has a span of 84 ft. and is a prestressed concrete structure built in 1953. The consulting engineers to the Stevenage Development Corporation for these structures are Messrs. Rendel, Palmer and Tritton.

Works in Bedfordshire and Huntingdonshire.

Work is proceeding on the Biggleswade by-pass which is 2¾ miles long and includes bridges over a main-line railway, a road, and the River Ivel. The scheme, the cost of which will be about £1,000,000, has been prepared for the Ministry of Transport by the Bedfordshire County Council, and is expected to be completed early this year. The contractors are Messrs. Sydney Green & Sons, Ltd.

The bridge at St. Neots is to be reconstructed in accordance with a design prepared by Sir Frederick Gibberd. The consulting engineers to the Ministry of Transport are Messrs. Posford, Pavry & Partners.

The work of modernising two miles of the Great North Road between Ellington Brook bridge and Alconbury has commenced, and is being carried out under the supervision of the Huntingdonshire County Council on behalf of the Ministry of Transport. The cost will be about £200,000. The contractors are Messrs. A. Monk & Co., Ltd.

Stamford By-pass.

A feature of the Stamford by-pass, which is 4½ miles long and was completed in the autumn of last year, is the comparatively large number of bridges; there are nine reinforced concrete bridges, two flood arches, three cattle-ways which are reinforced concrete box-culverts, and several ordinary culverts. The road has two 24-ft. carriageways comprising a 10-in. granular sub-base laid in two layers 4 in. and 6 in. thick respectively, an 8-in. cement-bound granular base, a 2½-in. course of hot-rolled gravel asphalt, and a 1½-in. wearing surface of granite asphalt. Where the ground is weak, the thickness of the sub-base is 14 in. The construction required the excavation of 840,000 cu. yd. of rock, clay, and other soils.

There are five types of bridge. The bridges over the rivers Gwash and Welland each comprise three spans and are fixed-arch structures; the facings are of Clipsham freestone. A bridge over a railway is a freely-supported beam-and-slab structure. The other bridges are continuous structures of beam-and-slab construction, the intermediate supports being a transverse row of cylindrical piers. The end supports are in some cases counterforted retaining walls as shown in (*Fig.* 5); for others the end supports are simple bench-seatings on embankments, as in the case of Sydney Farm bridge (*Fig.* 6). This bridge carries a main road over the by-pass, and has two spans each 57 ft. 6 in. long and two spans each 33 ft. long. The deck is of continuous cellular construction with an expansion bearing at one abutment, but is fixed at the other abutment and at the three piers.

The construction of Warren Road bridge (*Fig.* 7), which carries a farm road over the by-pass, comprises a two-hinge

Fig. 6.—Sydney Farm Bridge.

segmental portal frame having a clear square span of 99 ft. and a rise of 4 ft. 10 in. The depth at the crown is 3 ft. 6 in. The illustration shows a temporary hand-rail, but when completed the hand-rail will be similar to that shown in (*Fig.* 6). The reinforced concrete wing walls are faced with Clipsham stone. (*Figs.* 6 and 7 are reproduced from photographs taken by the "Stamford Mercury".)

The consulting engineers to the Ministry of Transport are Messrs. Stirling Maynard & Partners. The contractors were Messrs. A. Monk & Co., Ltd., and the sub-contractors for the excavation were Messrs. Shellabear, Price, Ltd. The total cost of the work, including four miles of road improvements beyond the by-pass, was about £1,500,000.

Grantham By-pass.

The Grantham by-pass, which is ten miles in length, is in course of construction and is expected to be completed in 1962. The new road will, with one exception, pass over or under all intersecting roads, and will require six reinforced concrete road bridges and two railway bridges. There will be about 1,500,000 cu. yd. of excavation.

The bridge shown in course of construction in (*Fig.* 8) is that by means of which the by-pass crosses the road from Grantham to Harlaxton. The structure, which has three main spans, contains forty-eight prestressed concrete beams which vary in weight from 9¼ to 32 tons each and which were erected by means of two 25-ton mobile cranes in five consecutive days. The illustration shows a beam 100 ft. long and weighing 22 tons being placed in position in the central span.

The work is being supervised on behalf of the Ministry of Transport by Lincoln (parts of Kesteven) County Council. The general contractors for the by-pass are Messrs. Robert McGregor & Sons, Ltd. The contractors for the bridges are Messrs. Simon-Carves, Ltd. The prestressed precast beams for the bridge in (*Fig.* 8) were supplied by St. Ives Sand & Gravel Co., Ltd.; the erection of the beams was carried out by Mobile Lifting Services, Ltd.

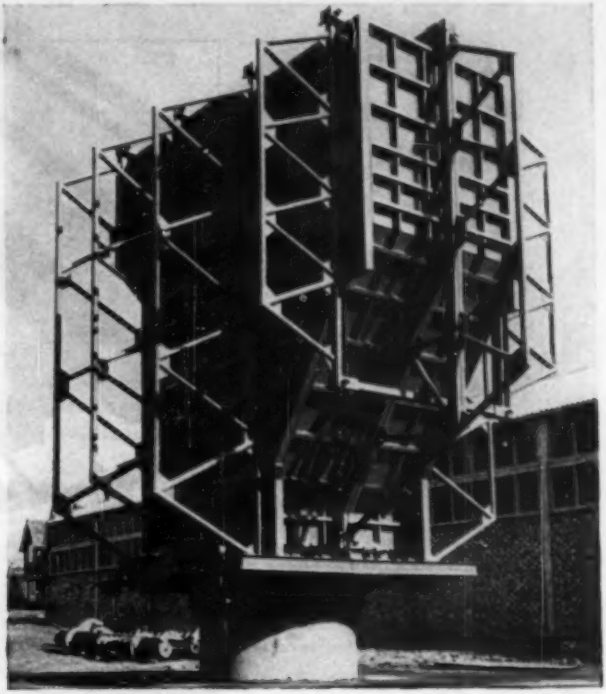
Works in Nottinghamshire.

One of the major constructional works in connection with the Great North Road in Nottinghamshire is the southern section of the Doncaster by-pass of which four miles are in this county and for which section the Nottinghamshire County Council are agents to the Ministry of Transport.

Other works include the proposed Newark by-pass which requires the con-

Fig. 7.—Warren Road Bridge.

Consultants: Sir William Halcrow & Partners
Contractors: The Demolition & Construction Co Ltd



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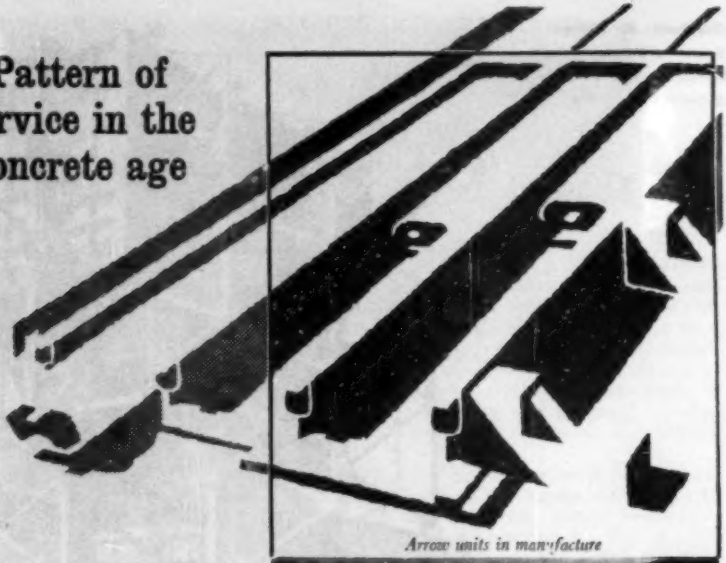

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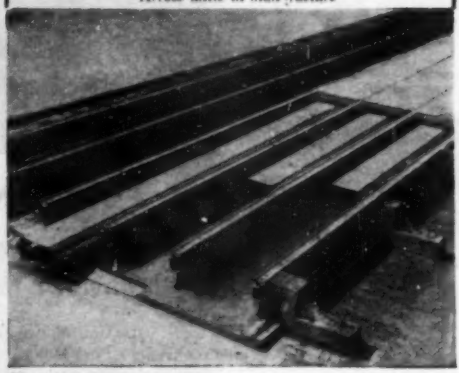

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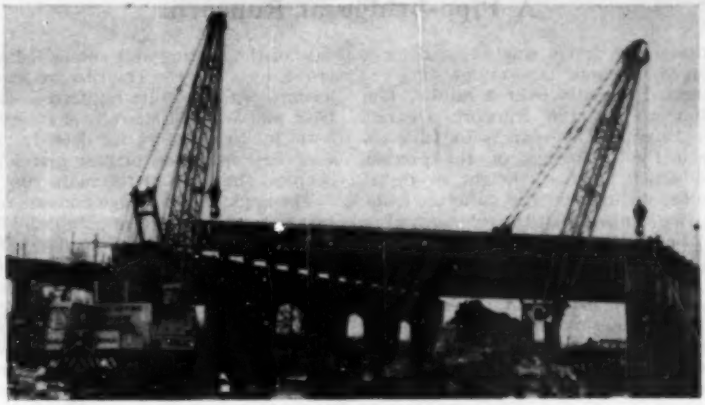

Fig. 8.—Bridge at Grantham By-pass.

struction of a viaduct across the River Trent in an area subjected to flooding. Information was needed for design purposes about the number of flood-openings which should be included in the viaduct. Originally nine 40-ft. flood-openings were proposed. Investigations made on models at the Hydraulics Research Station of the Department of Scientific and Industrial Research at Wallingford showed that six arches could be omitted without causing an appreciable increase in the flood levels. The elimination of six arches may save about £170,000.

The Blyth by-pass, which is two miles long, is complete, and connects with the Doncaster by-pass at the northern end. The work includes bridges to carry the new road over an existing main road and the River Ryton. The scheme, which cost about £340,000, was designed by the Nottinghamshire County Council for the Ministry of Transport. The contractors were Sir Robert McAlpine & Sons, Ltd.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Prestressed Concrete. By H. Kaylor. Technical College, Southampton. February 14. 7.15 p.m.; and Technical College, Newport, I.O.W. February 15. 7 p.m.

Surface finishes of Concrete. By F. Ward. West Wiltshire College of Further Education, Trowbridge. February 15. 7.15 p.m.; and Cambridgeshire Technical College, Cambridge. February 23. 7.30 p.m.

Prevention of Accidents in the Building Industry. By J. A. Hayward. Church Institute, Albion Street, Kingston-upon-Hull. February 16. 7.15 p.m.

Work Study in the Building Industry. By R. Geary. College of Technology, Portsmouth. February 16. 7.15 p.m.

The Essentials of Good Concreting. By E. E. H. Bate. Institute of Technology, Bradford. February 20. 7.15 p.m.

Structural Properties of Lightweight Aggregate Concrete. By G. H. Newton. West Wiltshire College of Further Education, Trowbridge. February 22. 7.15 p.m.

The R.I.B.A. Form of Contract. By D. Gardam. Mid-Essex Technical College, Chelmsford. February 23. 7.30 p.m.; and by I. N. D. Wallace. Sun Hotel, Chatham. February 28. 7.15 p.m.

A Pipe-bridge at Runcorn.

An interesting design was adopted for a reinforced concrete pipe-bridge (*Fig.* 1) constructed recently over a railway line at Runcorn for the Runcorn District Water Board. The span is 61 ft. 4 in. between the springings of the precast arches which rise to a height of 19 ft. above the rail-track. The structure comprises two semi-circular three-hinge arches each of which was precast in two parts (*Fig.* 2). The new bridge, which is 10 ft. away from an existing bridge, is to carry a 15-in. water main and a 6-in. brine main, and provision is made for another 6-in. brine main.

Each pair of ribs, which are 24 in. deep by 12 in. wide, were erected from each side of the cutting and connected at the crown by a hinge. The ribs are seated in a combined cast-insitu reinforced concrete base, and are held laterally at intervals of 8 ft. by tie-beams 5 in. deep by 12 in. wide cast insitu on precast pieces 2½ in. deep on the sides of the main ribs.

The deck of the bridge consists of precast concrete units 6 in. deep and 12 in. wide seated on the bottom flange of the main ribs and placed between the tie-beams. These units are covered with a concrete topping not less than 1½ in. thick. The pipe-supports were cast insitu in two rows. To comply with the safety requirements of British Railways, a closed non-metallic parapet of a height not less

Fig. 1.

Fig. 2.

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Fig. 3.

than 4 ft. 6 in. above the top of pipes was necessary and this was provided by 4-in. precast concrete units, the bottom flanges of which are bolted to the main ribs. The parapet units are shaped to suit the curve of the bridge, and on the outer faces have an exposed-aggregate finish of Criggion green granite (*Fig.* 3). The approaches to the bridge are flanked by reinforced concrete retaining walls 6½ in. thick.

The cost of the structure was about £3056. The bridge was designed for Mr. G. Penman, A.M.I.C.E., A.M.I.W.E., Engineer to the Runcorn District Water Board, by the British Reinforced Concrete Engineering Co, Ltd. The contractors were Messrs. Harry Fairclough, Ltd.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", FEBRUARY, 1911.

A Gasholder Tank.

"The first reinforced concrete gasholder tank in the United Kingdom was recently constructed at Leigh-on-Sea. As the site is on reclaimed foreshore, and is impregnated with salt water, a steel tank would have had to be protected with concrete, and as the cost of this would have been very high, it was decided to adopt reinforced concrete. The tank is 64 ft. in diameter by 29 ft. 6 in. deep. The wall of the tank is 8 in. thick at the bottom and 4½ in. thick at the top. The capacity of the tank is 538,000 gallons. The holder, for which the tank is required, has two lifts and a capacity of 160,000 cu. ft. The guide framing has nine standards, which are fixed on reinforced concrete buttresses incorporated with the tank. Inside the tank there are eighteen channel iron guides secured to the concrete wall with bolts. The wall is strengthened at these places with reinforced concrete buttresses."

A Long Prestressed Bridge in Germany.

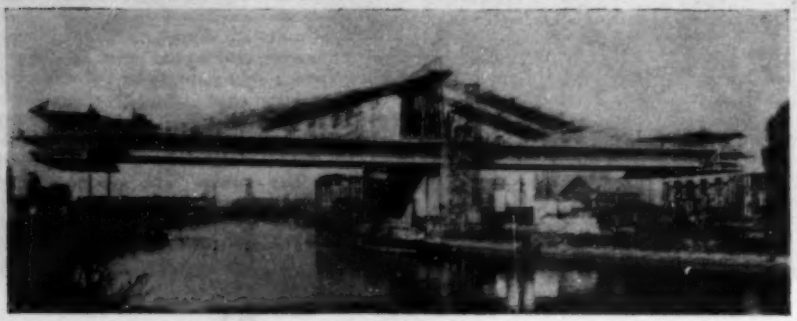

Fig. 1.

A PRESTRESSED concrete road bridge, said to be the longest of its kind in Germany, is being constructed as part of the express road system around Berlin. The bridge is 2970 ft. long, 95 ft. wide, and 43 ft. high, and will carry six lanes of traffic. There will be twelve spans from 213 ft. to 278 ft. long. The structure is of double box-girder construction, the vertical webs of the girders being 8 ft. 6 in. deep, the lower slab varying from 5 in. thick at the middle of a span to 2 ft. 8 in. thick at a pier, and the top slab being about 1 ft. thick. The piers are supported on 17-in. diameter prestressed piles of 65-tons capacity.

The bridge, which is of cast-insitu construction, is being built two spans at a time on the double cantilever principle from a pier (*Fig.* 1). Both cantilevers are extended about 12 ft. each week. The shuttering is cantilevered from the work already constructed. The concrete girders are prestressed about sixty hours after casting. After about 108 ft. of girder has been cast on each side of a pier, a temporary A-frame 32 ft. high is placed over the pier and cables from the top of the frame are used to support further extensions of the girder. The designer and contractors are Messrs. Dyckerhoff & Widman, in association with Beton & Monierban AG., and Messrs. Phillip Holzmann AG. The foregoing particulars and the accompanying illustration are abstracted from "Engineering News-Record".

Training Courses for Concrete Gangers.

THE National Federation of Building Trades Employers and the Federation of Civil Engineering Contractors have arranged a series of training courses during 1961 for concrete gangers. The courses will be held at the Cement and Concrete Association's Research Centre, Wexham Springs, on the following dates:

Course 1.—March 6 to 10 and March 20 to 24.

Course 2.—March 13 to 17 and April 10 to 14.

Course 3.—September 18 to 22 and October 9 to 13.

Course 4.—September 25 to 29 and October 16 to 20.

Course 5.—October 2 to 6 and October 23 to 27.

There will be a lecture and demonstration on prestressed concrete on March 24, at the conclusion of Course 1, to assess the demand for instruction in this subject. The fee for a course is £8 per week and accommodation can be provided at the training centre for £6 per week. Enrolment forms are obtainable from the Education Secretary, London Master Builders' Association, 47 Bedford Square, London, W.C.1.

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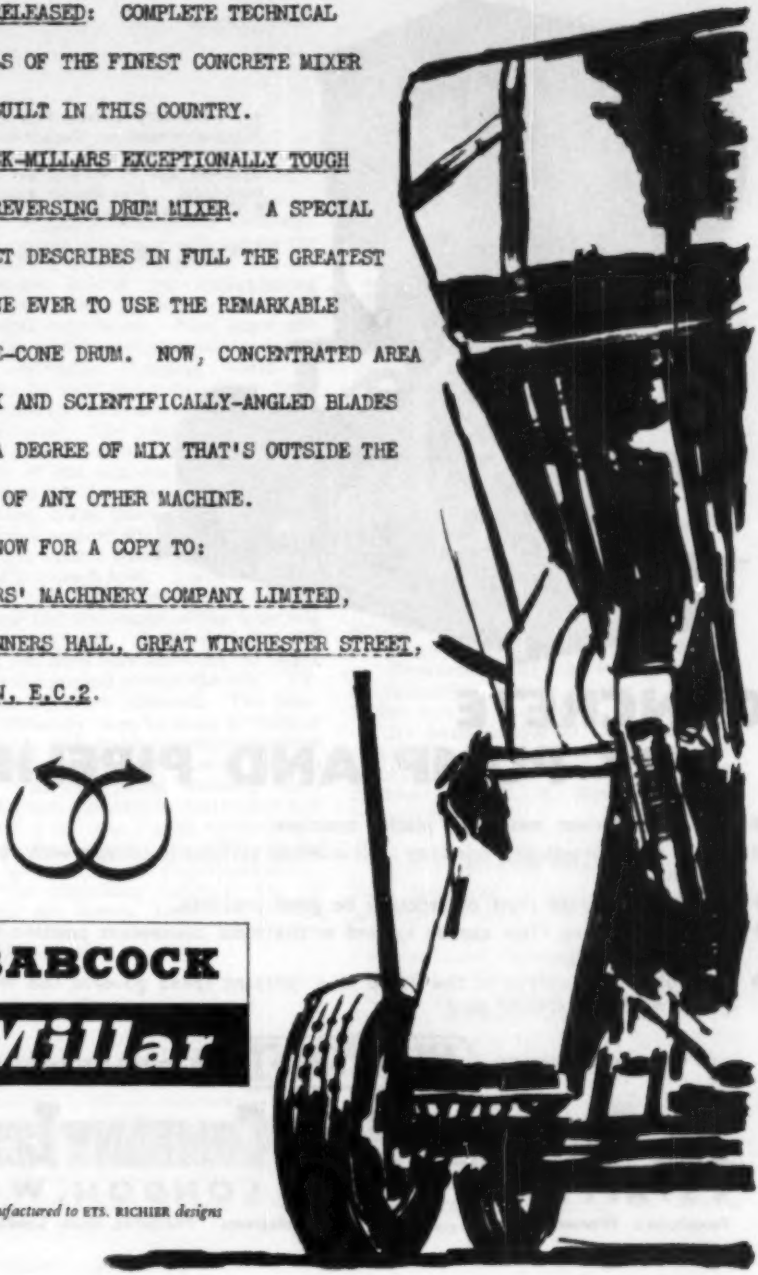

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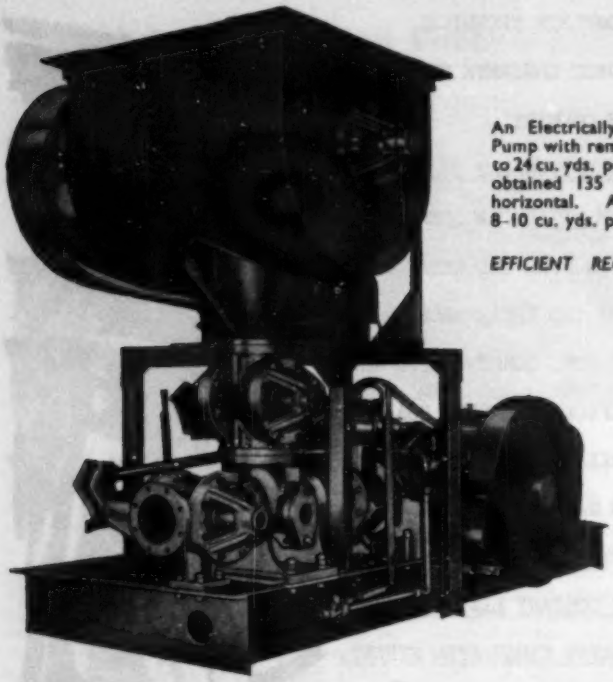

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A New Type of Deep Foundation.

THE method adopted for the construction of the foundations of two adjacent blocks of residential flats near Marble Arch, London, is unusual but is claimed to save time and materials. The structures are of thirteen and seven storeys respectively and have three-storey basements which made it necessary to provide additional support for adjacent roads and buildings. Ordinary underpinning would reduce the area available in the basements, and the obstructions caused by underpinning works would have prevented the use of mechanical excavators. Also, since the ground is mainly waterlogged sand and gravel, extensive pumping would be necessary to keep the excavations dry.

A method of chemical consolidation was therefore used. The process involves the mixture of fluids of low viscosity, the proportions of the mixture controlling the time taken for it to form a gel. Before excavating, holes lined with steel tube were bored around the periphery of the site and a steel I-beam 33 ft. long was dropped into each hole. The lower 10-ft. length of the beam was encased in concrete and the remainder of the hole was filled up with sand. A slurry of clay and cement was then injected to fill the larger voids in the ground around the site. The chemical injection followed. The holes were sufficiently close to cause a block of ground 5 ft. wide to be consolidated which sealed off the inflow of water to such an extent that one pump working intermittently was sufficient to deal with it and maintain a dry site during excavation.

The method is called the TDM process and is operated by the Cementation Co., Ltd. The consulting engineers for the buildings are Messrs. Jenkins & Potter, and the main contractors are Messrs. Percy Bilton, Ltd.

IMPERIAL COLLEGE OF SCIENCE AND TECHNOLOGY

BURSARIES IN CONCRETE STRUCTURES AND TECHNOLOGY

Bursaries of £460–£760 according to experience are available for Session 1961–1962. Candidates must hold a degree in engineering and have a good knowledge of the theory of structures. Full information from the REGISTRAR, IMPERIAL COLLEGE, London, S.W.7. Closing date 1st June, 1961.

Bulletins Received.

"**Sources of Road Aggregate in Great Britain.**" (London: H.M.S.O. Third edition, 1960. 6*s*.)

"**Studies in Composite Construction.**" By R. J. Mainstone. National Building Studies. Research Paper No. 28. Part III. Tests on the new Government offices, Whitehall Gardens. (London: H.M.S.O. 1960. Price 8*s*. 6*d*.)

"**Symposium on Effect of Water-Reducing Admixtures and Set-Retarding Admixtures on Properties of Concrete.**" Special Technical Publication No. 266. (Philadelphia: American Society for Testing Materials. 1960. Price 7.50 dollars.)

Cement and Concrete Association.

The bulletins in the following were published by the Cement and Concrete Association during 1960, and copies are obtainable free of charge from the Association at 52 Grosvenor Gardens, London, S.W.1.

"Specification Notes for Structural Reinforced Concrete in Buildings."

"Specification for Granolithic Floor Toppings Laid on In situ Concrete."

"Concrete Practice in Building Construction."

"Single Storey Industrial Buildings."

"Prestressed Concrete Pressure Vessels at Marcoule Nuclear Centre." By J. Bellier and M. Tourasse.

"An Investigation of the Stress Distribution in the Anchorage Zone of Post-tensioned Concrete Members." By J. Zielinski and R. E. Rowe. Research Report No. 9.

A Course on Site Labour.

THE Industrial Welfare Society have arranged to hold a two-day course on "Site Labour in the Building and Constructional Industries". The course which is to be held at Robert Hyde House, 48 Bryanston Square, London, W.1, on March 20 and 21, 1961, is designed primarily for the 150 firms in the constructional industries in the United Kingdom which each employ more than five hundred workers; it should, however, prove valuable to smaller firms. It will deal exclusively with the problems of managing manual labour on sites. Particulars can be obtained from the Administrative Officer, at the address given above.

DATA FOR PRICING REINFORCED CONCRETE.

MATERIALS (*Delivered London*).

AGGREGATES (per cu. yd.).

Washed sand: 26*s.* 9*d.* Graded gravel (¾ in.): 24*s.* 3*d.* Pit ballast: 25*s.* 6*d.*

CEMENT (per ton).

Ordinary Portland.—6 tons and upwards: 112*s.*; 1 ton to 6 tons: 124*s.*

Rapid-hardening Portland: As ordinary Portland plus 10*s.* 6*d.* "Aquacrete" and "417": As ordinary Portland plus 32*s.* 6*d.* "Colorcrete" (buff, red, and khaki): As ordinary Portland plus 60*s.* "Snowcrete" (white): 277*s.* 6*d.*

"Super-Cement": As ordinary Portland plus 32*s.* 6*d.*

High-alumina cement.—1 ton: 335*s.*; 2 tons: 325*s.* 6*d.*; 6 tons: 314*s.* 6*d.*

"Super Snowcem" waterproof cement paint: 100*s.* per cwt.

REINFORCEMENT (per ton ex stock including delivery in quantities from 10 to 500 tons).

Plain mild steel rods per B.S. No. 785.—1 in. and over: £50 5*s.*; ⅞ in. and ¾ in.: £51; ⅝ in.: £51 15*s.*; ½ in.: £53 7*s.* 6*d.*; ⅜ in.: £54 17*s.* 6*d.*; ¼ in.: £58 17*s.* 6*d.*

Hot-rolled deformed bars: As plain bars plus £1.

High-tensile (minimum yield point 44,000 lb. per sq. in.): As mild steel plus 19*s.* (For basic prices and allowances for quantities exceeding 500 tons and extras for quantities less than 10 tons, lengths greater than 40 ft. or less than 5 ft., bundling, labelling, and delivery in other areas, see "Concrete Year Book, 1961".)

Fabric per B.S. 1221A (per sq. yd.).—No. 109: 3*s.* ½*d.*; No. 121, 5*s.* 7½*d.*; No. 124: 3*s.* 4½*d.*; No. 125: 2*s.* 10*d.*; No. 126: 2*s.* 6½*d.*

TIMBER.—Sawn deal shutter boards: £95 to £100 per standard (165 cu. ft.); 1-in. boards: 1*s.* (av.) per sq. ft.; 1½-in. boards: 1*s.* 3½*d.* (av.) per sq. ft.

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Foundations (per cu. ft.).—3*s.* 6*d.*

Suspended slabs (per sq. yd.).—6 in., 18*s.*; 7 in., 21*s.*; 9 in., 27*s.*

Beams (per cu. ft.).—4*s.* Columns (per cu. ft.).—4*s.* 6*d.*

Walls (per sq. yd.).—6 in., 19*s.* 7*d.*; 9 in., 29*s.* 5*d.*; 12 in. and over (per cu. ft.), 3*s.* 6*d.*).

Extra (per cu. yd.) for rapid-hardening Portland cement. 1 : 2 : 4.—2*s.* 8*d.*

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⅞ in. and ¾ in.	74*s.* 6*d.*	80*s.* 3*d.*	77*s.* 11*d.*	—
⅝ in.	75*s.* 4*d.*	81*s.* 1*d.*	78*s.* 9*d.*	—
½ in.	82*s.* 10*d.*	89*s.* 7*d.*	87*s.* 4*d.*	—
⅜ in.	84*s.* 6*d.*	91*s.* 3*d.*	88*s.* 11*d.*	98*s.*
5/16 in.	86*s.* 8*d.*	93*s.* 5*d.*	91*s.* 2*d.*	103*s.* 8*d.*
¼ in.	87*s.* 9*d.*	94*s.* 6*d.*	92*s.* 3*d.*	104*s.* 9*d.*

Fabric per B.S. No. 1221A in flat areas (per sq. yd.) including straight cutting.—No. 109: 4*s.* 4*d.*; No. 121: 7*s.* 11*d.*; No. 124: 4*s.* 9*d.*; No. 125: 4*s.* 1½*d.*; No. 126: 3*s.* 6½*d.*

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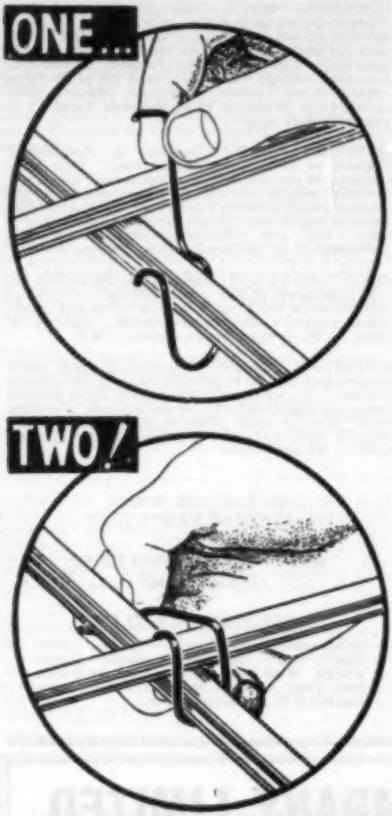
Walls and partitions (per sq. yd.): 19*s.* 10*d.*; ditto curved: 2£*s.* 7*d.*

Rectangular beams, lintels, columns, etc. (per sq. ft.): 2*s.* 8*d.*; ditto circular: 3*s.* 8*d.*



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SITUATIONS WANTED.

SITUATION WANTED. Young Lebanese structural engineer with two years' experience in reinforced concrete design (Member of A.S.C.E.; expecting M.Sc. in structural engineering next June) seeks employment with contractor or design office in Europe, Africa or the Middle East. Write to S. BALADI, Graduate School, University of Tennessee, Knoxville, Tenn., U.S.A.

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SITUATIONS VACANT. Reinforced concrete designers and detailers—your own bureau—no fees. Permanent positions. Call BUSINESS VACANCIES BUREAU, 29 Victoria Street, London, S.W.1. Telephone Abbey 7200.

SITUATION VACANT. A vacancy exists in a small design office in Bristol for a reinforced concrete designer-detailer of Higher National standard to assist the senior engineer. Work varied and interesting. Reinforced concrete design experience essential and high standard of draughtsmanship desirable. Five-days' week. Apply in writing to PROGRESSIVE STRUCTURES LTD., Berkeley Cottage, Berkeley Square, Bristol, 8.

SITUATION VACANT. Reinforced Concrete Engineers require Draughtsman for Project and Estimating Department at Head Office, Harrow. Write full details to BIERRUM & PARTNERS, 167 Imperial Drive, Harrow.

SITUATIONS VACANT. Designer-detailers and detailers required for interesting work in a happy office. Permanent appointments. Unusual methods of detailing taught. Fair pay and luncheon vouchers. M. M. KHAN, B.Sc., M.I.C.E., 88 Rochester Row, London, S.W.1. VICtoria 9858.

SITUATIONS VACANT. Reinforced concrete designers and detailers required for industrial and framed structures. Structural steelwork an advantage. Salaries £800–£1100 according to age and experience to right men, who need to be quick, competent and conscientious. Five-days' week. Luncheon vouchers. Interviews evenings and Saturdays. R. L. BOURGUI & PARTNERS, 85-87 Clarence Street, Kingston-upon-Thames. Telephone: Kingston 0677/0285/0323. Evenings: Elmbridge 2654.

SITUATIONS VACANT. Consulting engineers have vacancies for the following: Reinforced concrete designers with A.M.I.Struct.E.; Designer-detailers with H.N.C. or similar; Detailers with minimum two years' experience. The positions are permanent, with opportunity for advancement. Modern office and pleasant conditions. Salaries in accordance with experience. BYLANDER, WADDELL & PARTNERS, 169 Wembley Park Drive, Wembley.

SITUATIONS VACANT. Reinforced concrete detailers required by LEONARD & GRANT in their South London drawing offices. Only top men should apply. High salaries. Interview by appointment. Telephone: Tate Gallery 8496.

SITUATIONS VACANT. Structural and civil engineering draughtsmen required for Chichester office. Good prospects for promotion. Varied and interesting work. Apply in writing, stating age, experience and salary expected, to ARCHIBALD SHAW, B.Sc., M.I.C.E., 11 Little London, Chichester.

SITUATIONS VACANT. Applications are invited by a consulting engineer, from all grades of assistants, for a new office shortly to be opened in Streatham, Wimbledon or Richmond areas. Very pleasant working conditions. Bonus twice a year. Holidays as individually arranged. First-class knowledge required in respective grades of one of the following: In-situ, precast and prestressed concrete and structural steel, as applied to buildings, factories and industrial plants. Please state age, experience and salary required to BOX 4728, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Designer-detailer experienced in reinforced concrete required for London professional office. The work is varied, comprising all types of construction in concrete and steel, and offers scope for anyone interested in gaining design experience. Permanent position with excellent prospects. Five-days' week. Apply, stating full experience, to FARMER AND DARK, Romney House, Tufton Street, London, S.W.1.

SITUATION VACANT. Civil engineering assistant required by progressive and growing organisation for work in the office. The company is engaged on the design and construction of a wide variety of industrial and commercial projects throughout the country, and the vacancy arises through expansion. The successful applicant will be encouraged to obtain professional qualifications. Please send details of age, training, experience and approximate salary required to THE CHIEF CIVIL ENGINEER, INDUSTRIAL DEVELOPMENT & CONSTRUCTION LTD., Industrial House, Stratford-upon-Avon.

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A certain amount of technical training will be given, but progress with the Company will depend on personal initiative and managerial ability as complete control of the works will be vested largely in the manager.
Initial appointment in the range up to £1500 p.a. with bonus, pension scheme and Company car.
Write for interview in the first instance with brief personal history to BOX 4726, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced Concrete Designer/draughtsman required, O.N.C. or H.N.C. an advantage, also a qualified or post-graduate engineer required to work on design and detailing of precast concrete frames, including multi-storey flats. Good salary, prospects of promotion, permanent position, staff pension fund and profit-sharing scheme. Apply TECHNICAL DIRECTOR, CONCRETE (MIDLANDS) LTD., Dovehouse Fields, Lichfield, Staffs. Telephone: Lichfield 3555.

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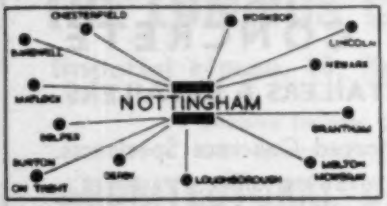

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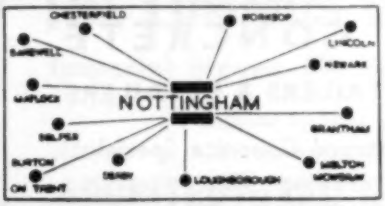

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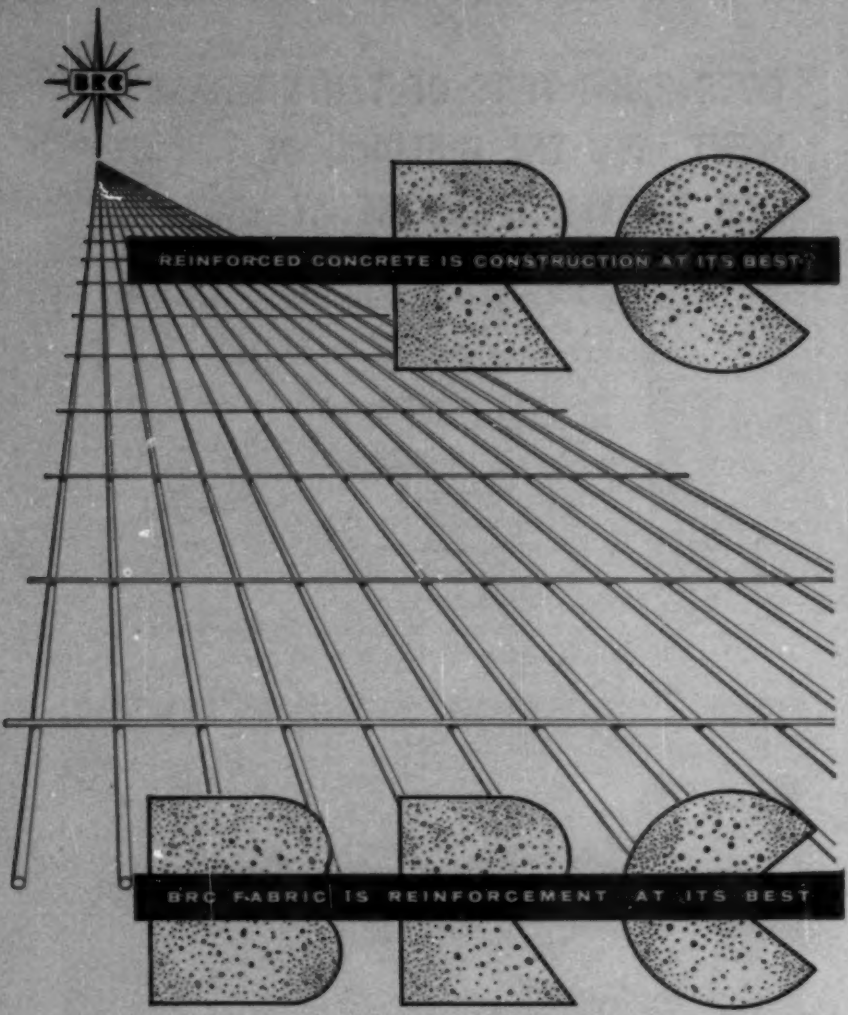

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